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ANALYSIS AND DESIGN PROCEDURES FOR PILE FOUNDATIONS SUPPORTING TEMPORARY TOWERS SKYWAY STRUCTURES

SAN FRANCISCO-OAKLAND BAY BRIDGE EAST SPAN SEISMIC SAFETY PROJECT

Prepared for

CALIFORNIA DEPARTMENT OF TRANSPORTATION

March 2001

Earth Mechanics

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March 5, 2001
Project No. 98-42-0054

California Department of Transportation
Engineering Service Center
Office of Structural Foundations
5900 Folsom Boulevard
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Attention: Mr. Mark Willian
Contract Manager

Analysis and Design Procedures for
Pile Foundations Supporting Temporary Towers, Skyway Structures
SFOBB East Span Seismic Safety Project

Dear Mr. Willian:

The geologic and geotechnical studies for the San Francisco-Oakland Bay Bridge (SFOBB) East Span Seismic Safety Project are being conducted by Fugro-Earth Mechanics (a joint venture of Fugro West, Inc., and Earth Mechanics, Inc.) under California Department of Transportation (Caltrans) Contract 59A0053. This report was prepared as a part of the work scope authorized by Task 5 of the referenced contract.

The design drawings for the Skyway Structures indicate that temporary towers are likely required for support of the peripheral deck sections of the structure during construction. The design and performance of temporary structures during construction is solely the responsibility of the contractor. However, in view of the relative importance of this structure, the unique geologic conditions at the site, and the relatively large loads to be supported by the temporary towers, Caltrans requested Fugro-EM to prepare this report as guidance to the contractor in the form of criteria and example calculations.

To assist with the preparation of the project's special provisions, this report was submitted in draft on July 12, 2000. Subsequently, location-specific subsurface explorations were performed at the temporary tower locations. The draft report has been updated to include subsurface cross sections based on those additional explorations. However, the example calculations presented previously have not been updated to reflect the variations from conditions previously assumed revealed by the subsequent subsurface explorations.

Earth Mechanics

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California Department of Transportation
March 5, 2001 (Fugro 98-42-0054)

This report is organized into seven sections of text, supporting plates, and five appendices. The text sections provide site characterization information and summarize analyses and design procedures. Detailed results of the example calculations are presented in the appendices.

On behalf of the project team, we appreciate the opportunity to contribute to Caltrans' design of the new bridge to replace the existing SFOBB East Span. Please call if we can answer any questions relative to the information presented in the enclosed report.

Sincerely,

FUGRO-EARTH MECHANICS, A Joint Venture

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SAN FRANCISCO-OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT

CALTRANS CONTRACT 59A0053

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MARCH 2001

Prepared For:

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1.0 INTRODUCTION

1.1 TEMPORARY TOWER LOCATIONS

The 100% Design Submittal (TY Lin/M&N, 2000) for the Skyway structures indicates that temporary towers will be required during construction to support portions of the superstructure. The illustrations presented on Construction Sequence Nos. 1 and 5 on TY Lin/M&N (2000) show the anticipated locations of three sets of temporary towers, designated as Locations A, B and C. The approximate location of the temporary towers derived from those drawings is shown on Plates 1 and 2. As shown on those plates, each set of temporary towers includes a tower supporting the eastbound (E) structure and a tower supporting the westbound (W) structure. The individual towers, therefore, are identified by two character labels that designate the location (i.e., A, B, or C) and the structure (i.e., E or W). For example, temporary tower AE supports the eastbound structure at Location A. Locations A and B are between Piers E2 and E3 (where the Skyway connects to the Main Span) and Location C is between Piers E16 and E17 (where the Skyway connects to the Oakland Shore Approach).

1.2 BACKGROUND

The design of temporary structures during construction is solely the responsibility of the contractor. However, in view of the relative importance of this structure, the unique geologic conditions at the site, and the relatively large loads to be supported by the temporary towers, consideration was given to requiring a completed design of the temporary towers. Caltrans eventually decided that, in view of the tight schedule specified for the completion of design, it would not be feasible for TY Lin/M&N (a joint venture of TY Lin International and Moffatt & Nichol, Engineers) to design the temporary towers. However, in view of the complicated geology at the site, Caltrans also decided that substantial guidance should be provided to the contractor in the form of criteria and example calculations for the design and installation of pile foundations.

To assist with preparation of the project's special provisions, the recommendations presented in this report were submitted in draft on July 12, 2000. At that time, no location-specific geotechnical exploration had been performed at the anticipated temporary tower locations. However, in view of the schedule constraints of the project, Caltrans instructed Fugro-EM (a joint venture of Fugro West, Inc., and Earth Mechanics, Inc.) to develop typical soil profiles on the basis of the geotechnical data available at adjacent pier locations, and to perform example calculations to illustrate foundation design procedures on the basis of those profiles. Subsequently, marine CPT soundings were performed at each of the temporary tower locations during the Phase 3 exploration program. Interpretations of the conditions revealed by those explorations are described in Section 2.0 of this report. *However, since the example calculations are provided only as illustrations of design procedures, they have not been updated or modified to reflect the variations between the previously generated typical soil profiles and conditions revealed by the location-specific explorations.*

1.3 PILE TYPE SELECTION

The selection of pile type, pile length, and pile section for the support of the temporary towers is the responsibility of the contractor. The temporary tower locations, however, are underlain by a significant thickness of clay strata. During driving in clay soils, the clay surrounding a pile is remolded and positive excess pore water pressures are generated. As a result, the pile capacity at the end of driving normally will be less than the ultimate static pile capacity used for design. As the pore pressures dissipate, the pile capacity will increase. This phenomenon, which also is observed in some fine-grained sand layers, is referred to as setup. In general, the setup period is proportional to the diameter of the driven pile. For very-large-diameter piles, field measurements have shown that the time required for piles to regain their ultimate strength in clay soils can be on the order of several months to years. Consequently, to minimize impacts to cost and schedule, specialized pile acceptance criteria that are different from those specified in the Caltrans Standard Specifications (Caltrans, 1995) were developed for the large-diameter piles supporting the Skyway structure piers. Since design and construction of those large-diameter, high-capacity foundations are relatively unconventional and would require substantial review, Caltrans construction personnel expressed the desire that more conventional pile acceptance criteria be applicable to the piles supporting temporary structures. Therefore, they suggested that the maximum allowable capacity for piles supporting the temporary towers be limited to 1.8 meganewtons (MN) (200 tons).

It is recommended that piles for temporary tower foundations be: 1) steel pipe piles, 2) precast-prestressed concrete piles, 3) steel H-piles, or 4) precast-prestressed concrete shells. Cast-in-place piles are not recommended for consideration as temporary tower foundations. Further, it is recommended that pile foundations have a maximum sectional dimension on the order of 0.61 meter.

1.4 PURPOSE AND SCOPE OF WORK

This report documents the geotechnical analyses and recommendations by Fugro-Earth Mechanics (a joint venture between Fugro West, Inc., and Earth Mechanics, Inc.) provided to date for the Skyway Structure Temporary Towers. The analyses and recommendations presented in this report are based primarily on: 1) the information obtained from geologic site investigation and soil laboratory testing as described in the Final Marine Geotechnical Site Characterization report (Fugro-EM, 2001a,b), and 2) the design information provided by TY Lin/M&N and Caltrans.

As requested by Caltrans, this report includes:

- Subsurface cross sections at the location of each of the temporary towers,
- A number of example calculations for a range of pile types and sizes, and

- Recommended criteria for the design and construction of temporary tower pile foundations.

1.5 ANALYSES PERFORMED

Four different types of analyses are described in this memorandum, although not all have been performed for all potential pile types and locations. The procedures are discussed in detail in the subsequent sections, but can be grouped as follows:

- Ultimate Static Axial Pile Capacity
- Lateral Load Capacity and Load-Deflection
- Axial Load-Deflection
- Drivability Assessment

Three different pile types have been selected as being typical of the piles that may be used, two of which have been analyzed for two pile sizes:

- Steel Pipe Piles (0.41- and 0.61-meter-diameter)
- Precast Square Concrete Piles (0.41- and 0.61-meter-square)
- Steel "H" Piles (0.36-meter width)

As described above, preliminary typical soil profiles were used as the basis for the above analyses. Therefore, it should be recognized that the results presented above do not reflect the subsequent location-specific subsurface explorations and should likely not be used for design. Final design should be done by the contractor once the temporary tower structure type and foundation loads are known.

The input parameters and results for example calculations at each of the three tower locations are presented on a series of plates in Appendices A through E, the nomenclature for which is used for subsequent appendices and is as follows:

Plate A1-X.Y

where:

- "A" is the analysis type designation (also the appendix letter),
- "1" is the plate type (there is only one type for the axial pile capacity section),
- "X" is the tower location (A, B or C), and
- "Y" is the sequential number for the particular analysis type and tower location.

For example, A1-A.1 presents Preliminary Axial Pile Design Parameters and Example Results for Location A and the first plate in the series.

2.0 SUBSURFACE STRATIGRAPHY

2.1 PRELIMINARY SOIL PROFILES

The approximate location of the temporary towers, with respect to the piers and borings, is shown on Plates 1 and 2. At the time this report was prepared in draft, little location-specific data were available. Therefore, for the purposes of developing the special provisions and preparing illustrative example calculations, typical soil profiles at the locations of the temporary towers were developed as follows:

- Location A - Boring 98-26 to 82.8 meters below mudline
- Location B - Boring 98-19 to 37.5 meters and then Boring 98-27 from 37.5 meters to 90.6 meters below mudline
- Location C - Boring 98-39 to 104.2 meters below mudline

The eastbound and westbound bridge alignments were not differentiated when developing the preliminary typical soil profiles or performing subsequent analyses.

2.2 PHASE 3 SITE INVESTIGATIONS

During the Phase 3 exploration program, eight CPT soundings were performed in the anticipated temporary tower locations. The locations of those soundings are also shown on Plates 1 and 2. As shown on those plates, three CPT soundings were performed at Location A, three more at Location B, and an additional two at Location C. Logs of the CPT soundings are presented in the Final Marine Geotechnical Site Characterization report (Fugro-EM, 2001a).

2.3 SUBSURFACE CROSS SECTIONS

To illustrate the subsurface stratigraphy at the various tower locations four subsurface cross sections were prepared and are presented on Plates 4 through 7. The location of the section lines are shown on Plates 1 and 2, and a key to the data presented on the cross sections is provided on Plate 3. The cross sections at Locations A and B (A-A' and B-B') are oriented perpendicular to the N6 alignment, while the cross sections at Location C are along the centerline of the eastbound (CE-CE') and westbound (CW-CW') alignments. The sections have no vertical exaggeration and include profiles of the Phase 3 CPT soundings as well as previous borings. The cross sections extend down to elevation (El.) -75 meters. Also shown for each CPT sounding and boring are the interpreted soil shear strengths derived either from CPT tip resistance or from strength testing of samples from borings.

2.4 STRATIGRAPHIC CONDITIONS

The stratigraphic conditions underlying each temporary tower location are illustrated on Plates 4 through 7. The primary geologic units identified within the depth range of interest are:

- Young Bay Mud (YBM),
- Merritt-Posey-San Antonio (MPSA) Formations,
- Old Bay Mud (OBM), and
- Upper Alameda Marine (UAM)

Detailed descriptions of the various geologic units and their material properties are provided in the Final Marine Geotechnical Site Characterization Report (Fugro-EM, 2001a). The following is a summary of the variations in shallow conditions illustrated on the cross sections:

- At Locations A, B, and C, the thickness of Young Bay Mud (YBM) generally increases to the north. This observation is generally consistent with the identification of the N6 alignment along the flank of an east-west-trending YBM-infilled paleochannel. In general, the Old Bay Mud (OBM) crust also slopes to the north, which suggests that the YBM-infilled channel may be nested within older paleochannels.
- At Location A, the presence of sand layers within the Merritt-Posey-San Antonio (MPSA) Formations appears to increase to the north, suggesting that the sand may have infilled a deeper paleochannel. In general, little sand is present beneath Tower AE, while up to 3 meters of sand are present at the location of Tower AW.
- At Location B, the subsurface stratigraphy generally includes a greater thickness (on the order of up to 8 meters) of relatively dense sand layers than at Location A. The sand layers are present beneath both Towers BE and BW. Since the surface of the top of the Old Bay Mud (OBM) is also channeled, the thickness of stiff clay layers of the MPSA also increases to the north. Very stiff clay layers of the OBM are encountered approximately 8 to 9 meters deeper at the northern edge of the N6 alignment than at the southern edge.
- An interlayered sequence of sand, silt, and clay within the Upper Alameda Marine (UAM) sediments appears to be present below approximately El. -60 meters at both Locations A and B.
- Temporary Tower C appears to be located in an area of intersecting east-west- and northwest-southeast-trending paleochannels. In general, the Young Bay Mud (YBM) is approximately 3 to 4 meters thicker beneath Tower CW than beneath Tower CE. Relatively dense sand layers of the Merritt-Posey-San Antonio (MPSA) Formations that are present beneath Tower CE appear to be have been eroded beneath much of

Tower CW as well. However, the lower sections of YBM appear to include a relatively continuous sand layer at elevations similar to the MPSA sand layers farther to the south. The sand layers within the YBM, however, appear to be of a somewhat lower relative density.

- At all locations, although typically very stiff, the Old Bay Mud and Upper Alameda Marine sediment sequences frequently include overconsolidated crusts that are relatively flat-lying, within which the shear strength of the materials is on the order of 50 to 150 kilopascals (kPa) greater than the materials underlying those crusts.

3.0 AXIAL AND LATERAL PILE CAPACITY AND LOAD-DEFORMATION CONSIDERATIONS

3.1 ULTIMATE STATIC AXIAL PILE CAPACITY

All three of the preliminary soil profiles combined with each of the selected pile types and sizes have been analyzed for ultimate static axial pile capacity. Axial capacity is likely to govern the pile length, and so the use of all three profiles was considered appropriate to assess the variability in required pile penetrations.

Information provided in the draft 100% Design Submittal (TY Lin/M&N, 2000) drawings indicates that the temporary towers will have design loads of between 10 and 15 MN. As the pile configuration and number of piles are unknown, the suggested 1.8-MN maximum allowable load has been assumed as an indicative design load for each pile. Assuming that the axial load is distributed equally between the piles supporting the structure and ignoring the self weight of the temporary tower, a minimum of approximately six to nine piles will be required per temporary tower.

3.1.1 Factor of Safety

As discussed subsequently, the axial pile deflection required to mobilize the end-bearing component of pile capacity is significantly greater than the deflections required to mobilize the skin friction component of pile capacity. To reduce the potential for excessive axial deformation of the temporary towers, it is recommended that the end-bearing component of axial capacity be neglected for service load design. Additional pile capacity from end bearing (albeit at larger pile deflections) will be in reserve for extreme infrequent dynamic loads (i.e., extreme earthquake loads). A factor of safety of at least 2.0 against service loads is commonly used during the design of pile foundations. However, since the towers are temporary structures, a factor of safety of 1.5 against service loads may be appropriate for design. On the basis of the above recommendations, the ultimate capacity (from skin friction alone) required for the maximum 1.8-MN pile is 2.7 MN.

3.1.2 Design Methodology

The methods used for analysis were adapted from the American Petroleum Institute (API) recommendations given in the API RP 2A Guidelines for Design of Fixed Offshore Structures (API, 1993a,b). The API guidelines were used and the design equations modified to reflect the site-specific soil conditions. The API methods are considered to be particularly suited to the primarily marine foundation soils encountered at the temporary tower locations.

In the API method, the ultimate axial compressive capacity (Q) for a given penetration is taken as the sum of the skin friction on the pile wall and the end bearing on the pile tip. As discussed above, it is recommended that end-bearing capacity not be considered for service load design. The end-bearing component should, however, be considered for the development of axial load-deformation relationships and for structural design under seismic loads.

3.1.3 Example Calculations

The input parameters and results for each of the three tower locations are presented on a series of plates in Appendix A. These plates present the soil type, strata unit designation, and parameters used in the calculations in addition to the calculated unit skin friction, unit end bearing, and the ultimate axial skin friction capacity curves (presented with no factor of safety). On these plots, the ultimate compressive capacity is shown as a solid line and the ultimate tensile capacity is shown as a dashed line.

The following table shows the estimated pile lengths to achieve the 2.7-MN ultimate skin friction load capacity for each pile type and tower location.

	Example Required Pile Penetration (meter)		
Pile Type	**Location A (Boring 98-26)**	**Location B (Borings 98-19 + 98-27)**	**Location C (Boring 98-39)**
0.41-Meter-Diameter Steel Pipe Pile	38.7	39.5	34.1
0.61-Meter-Diameter Steel Pipe Pile	30.8	31.2	26.8
0.41-Meter-Square Precast Concrete Pile	33.5	34.0	29.6
0.61-Meter-Square Precast Concrete Pile	27.4	27.5	23.5
0.36-Meter Steel H-Pile	35.4	36.8	31.4

3.2 LATERAL PILE LOAD-DEFLECTION (p-y) RELATIONSHIPS

Lateral load-deflection (p-y) curves were generated to assist with the evaluation of the lateral load-deflection behavior of the temporary tower structures. Those p-y curves were then used in a finite-difference analysis, where the soil is modeled as a series of non-linear springs that, when combined with the flexural rigidity of the pile, resist shear loading applied at mudline. Lateral load-deflection (p-y) curves were developed for each of the five pile types and for two typical soil profiles (tower locations A and C).

3.2.1 Analytical Methods

Static and Seismic p-y Curves. The procedure used to generate the static p-y curves was generally based on the recommendations of API (1993a,b), which provides guidelines for developing p-y curves for different soil materials.

Recommendations for strain-softening p-y curves to evaluate piles under cyclic loading conditions also are presented in API (1993a,b). However, those recommendations that represent an envelope of maximum pile-resistances that degrade with additional cycles of loading are generally applicable to pseudostatic push-over analyses. The API recommendations are, therefore, considered to be unsuitable for time-history-based, soil-structure interaction analyses for earthquake loading conditions. For time-history-based, soil-structure interaction analyses, it is recommended that a p-multiplier of 0.5 be used to model both cyclic degradation and group effects.

Depths for Generation of p-y Curves. The choice of the depths at which p-y curves should be produced is significant. As a minimum, curves should be generated at every change in soil parameter (i.e., undrained shear strength, effective friction angle, unit weight, and each reference strain value). Additionally, p-y curves should likely be spaced no farther apart than 1 pile diameter for the upper 20 pile diameters. Curves should also be generated at points immediately above and immediately below the "critical depth" as defined in API (1993a,b).

Scour Considerations. The effects of scour around a pile are twofold:

1. A general scouring of the mudline soils reduces the effective stresses at any given depth in the soil, leading to a reduction in assigned skin friction and a reduction in ultimate lateral soil resistance.
2. Local scouring or "slotting" around the pile reduces the length of pile in contact with the soil, thereby reducing capacity.

Long-term scour around the existing pier structures ranges from 1 to 5 meters according to recent surveys. However, for the design of temporary structures supported on relatively small piles, the effect of scour will be less significant. For final design of the temporary towers, it is recommended that a general scour of 0.5 meter and an additional local scour equal to one pile diameter be used. Scour considerations have not been included in these specimen calculations, but it is recommended that they are considered carefully prior to finalizing the design. For the evaluation of stresses induced in structural members during seismic loading, sensitivity analyses that both include and ignore scour considerations should likely be performed.

3.2.2 Example Calculations

Example p-y curves generated using the above recommendations are presented on a series of six plates (Plates B1-A.1 through B1-A.3 and B1-C.1 through B1-C.3). It should be noted that these plates only indicate the ultimate loads (p_u or p_n) and critical displacements (y_c or y_u), and that the remainder of the curves should be scaled from these values using the appropriate normalized values provided in API (1993a,b).

The above p-y curves were incorporated into a finite difference analyses using the computer program LPILE (Ensoft, 1997) to evaluate the lateral load-deflection response of the pile at the mudline. The results of those evaluations are given on Plates B2-A.1 through B2-A.5 and B2-C.1 through B2-C.5. Each of the five chosen pile types have been analyzed under "fixed" pile head (restrained against rotation) and "free" pile head (unrestrained against rotation) conditions. The "true" behavior of the pile head depends on the degree of fixity provided by the pilecap, and will likely lie within these two extremes. It should be noted that these fixity conditions apply to a pile flush with the bay floor and a lateral load applied at that level. Consideration should be given to the effect of any free-standing pile length ("stick-up") above bay floor, or a different loading position. Furthermore, if the predicted deflection under the working lateral loads is significant, then the additional bending moment and shear stress applied by the eccentric axial load should also be included.

For a given pile width or diameter, the p-y curves are identical. The lateral load-deflection response of the pile head will vary depending upon the bending stiffness of the pile, which is a function of its geometry and material type. For example, the difference in the behavior of the 0.41-meter-diameter steel pipe pile and the 0.41-meter-square concrete pile can be seen by comparing Plates B2-A.1 and B2-A.3.

3.3 AXIAL PILE LOAD-DEFLECTION RELATIONSHIPS

3.3.1 Load-Transfer Considerations

In addition to load capacity, the design of driven piles should consider axial load-deflection characteristics. As discussed above, the end-bearing component of pile capacity has not been included in the assessment of ultimate pile capacity for these temporary structures. The main reason for this is that the pile deflection required to mobilize the end-bearing component of pile capacity is typically significantly greater than the deflections required to mobilize the skin friction component of pile capacity. It has long been recognized (in offshore platform design) that the component of pile capacity due to end bearing is largely in reserve for piles driven through primarily clay soils and designed for normal factors of safety. In other words, the skin friction typically exceeds the applied load (i.e., the ultimate load divided by the factor of safety). Thus, the majority of the applied service load is mobilized in skin friction along the pile shaft at levels of deflection much smaller than those required to mobilize significant components of the end bearing. Although the end-bearing component of pile capacity has not been included in the assessment of ultimate pile capacities, it should be considered for the development of axial load-deformation relationships and for structural design under seismic loads.

3.3.2 Analytical Methods

API (1993a,b) provides guidelines for developing t-z and q-z curves for different soil materials. Direct use of the API guidelines is recommended for assessing end bearing (q-z) curves and side shear (t-z) curves for sand and clay.

3.3.3 Example Calculations

The example t-z curves have been evaluated using the methods described above at depths corresponding to changes in soil parameters sufficient to fully describe the ultimate unit skin friction profile. These curves have been generated for two soil profiles (Locations A and C) for the five specimen pile types and lengths. The normalized curves are presented on Plates C1-A.1 through C1-A.5 and C1-C.1 through C1-C.5. It should be noted that the "t" values are expressed in terms of load per unit area (kPa) and need to be multiplied by the pile perimeter and the incremental length (along the axis of the pile) to estimate the total force that can be mobilized in each t-z spring. In addition, the effects of scour were ignored for these analyses.

In order to evaluate the axial pile head load-deflection response, these curves were incorporated into an analytical model that assesses the combined behavior of the pile and soil to an applied axial load. In such models, the axial compression of the pile is coupled with the progressive mobilization of the non-linear side springs. As an increasing load is applied to the head of the pile, it deflects axially downwards and, initially, the upper springs are mobilized. As the pile head deflection increases, load is transferred onto the lower springs and eventually the end spring (q-z curve) is mobilized, if it is present. The ultimate axial load capacity is reached when all the springs are mobilized. Example pile head load-deflection curves are presented on Plates C2-A.1 through C2-A.3 and C2-C.1 through C2-C.3 for the side friction only (tension) case. The effect of including the end (q-z) spring is illustrated on a comparable set of Plates C3-A.1 through C3-A.3 and C3-C.1 through C3-C.3.

In order to appreciate the effect of the compressibility of the pile, it should be noted that if the pile were completely rigid, then each of the t-z springs would be fully mobilized simultaneously. In this case, the ultimate resistance would be reached at a pile head deflection equal to one percent of the pile diameter.

3.4 PILE GROUP ACTION

The combined behavior of closely spaced piles will be different from that of the sum of the individual piles. The interaction between the piles should be considered if the piles are closer together than 8 or 10 pile diameters. If the spacing is less than 3 pile diameters, then the interaction may be very significant in terms of ultimate capacity and lateral and axial load-deformation response. Further details on evaluating the magnitude of this effect are given in API (1993a,b), Section 6.9.

4.0 PILE DRIVABILITY CONSIDERATIONS

To evaluate the constructibility of the proposed pile foundations, drivability analyses should be performed to assess the pile details (wall thickness, reinforcement, etc.) and equipment required to successfully install the piles to their target penetration without overstressing them. The following sections summarize:

- Methodology and conditions to be considered in drivability evaluations, and
- Example results of drivability analyses for the temporary towers.

Drivability analyses have been performed based on the available soils data and anticipated pile sizes and target penetrations. For the purposes of demonstrating the analysis required, one hammer has been selected to drive two typical pile types through a single soil profile. The combination chosen was the Delmag D62 driving steel pipe piles and square precast concrete piles at Location B. For final design and hammer selection, it is recommended that drivability analyses be performed using the final soil profiles to confirm that the selected piles can be driven to their design penetration without overstressing them.

4.1 DRIVABILITY ANALYSES

4.1.1 Soil Resistance to Driving

Computation of the soil resistance to pile driving is analogous to the computation of ultimate axial pile capacity by the static method. The resistance to driving is the sum of the shaft resistance and the toe resistance during driving. The shaft resistance is computed by multiplying the average unit skin friction during driving by the embedded surface area of the pile. The toe resistance is computed by multiplying the unit end bearing by the end-bearing area. Unlike static pile capacity computations, end bearing is not limited to the frictional resistance developed by the soil plug.

For the purpose of these preliminary evaluations, the methods suggested by Stevens et al. (1982) were used to calculate soil resistance to driving. For large-diameter pipe piles, Stevens et al. (1982) recommend computing lower- and upper-bound values of soil resistance to driving for both coring and plugged pile conditions. For concrete displacement piles, only the plugged cases are considered. When a pipe pile cores, relative movement between pile and soil occurs both on the outside and inside of the pile wall. Shaft resistance is, therefore, developed on both the outside and inside pile wall. For the coring condition, the end-bearing area is equal to the cross-sectional area of steel at the driving shoe. When a pipe pile plugs, the soil plug moves with the pile during driving. For the plugged condition, shaft resistance is mobilized only on the outer wall, and the end-bearing area is the gross area of the pile.

Whether or not a pipe pile is coring, partially plugged, or plugged is determined by the soil conditions, pile diameter, pile roughness, and pile acceleration during driving. Plugging during continuous driving in predominantly cohesive soils is unlikely, as discussed by Stevens (1988). The piles are assumed to initially plug, however, after each add-on is made or during significant delays. For a coring pile, a lower bound is computed assuming that the skin friction developed on the inside of the pile is negligible. An upper bound is computed assuming the internal skin friction is equal to 50 percent of the external skin friction. For a plugged pile, a lower bound is computed using unadjusted values of unit skin friction and unit end bearing. An upper-bound plugged case for granular soils is computed by increasing the unit skin friction by 30 percent and the unit end bearing by 50 percent. A corresponding increase in limiting values for unit skin friction and unit end bearing is assumed. For cohesive soils, the unit skin friction is not increased and the unit end bearing is computed using a bearing capacity factor of 15, which is an increase of 67 percent.

The lower- and upper-bound soil resistance curves represent experience gained from previous projects. That experience with the drivability of large-diameter steel pipe piles into predominantly clay soils indicates that the soil resistance calculated for a "coring" pile will generally provide the best estimate of the field blow counts, provided the pile wall is of sufficient thickness to effectively transmit the energy provided from an adequately-sized pile driving hammer.

With the exception of clay skin friction, the unit skin friction and unit end-bearing values used in the drivability analyses are the same as those used to compute static pile capacity. For piles driven in cohesive soils, the unit skin friction during continuous driving is computed using the stress history approach presented by Semple and Gemeinhardt (1981). The unit skin friction for static loading is first computed by using the C.6.4.2 method recommended by the American Petroleum Institute (API, 1986). The unit skin friction for static loading is then adjusted incrementally by multiplying it by a pile capacity factor, such that:

$$f_{dr} = F_p f$$

where: f_{dr} = unit skin friction used in the drivability analyses,
F_p = an empirical pile capacity factor, and
f = unit skin friction for static loading conditions.

The pile capacity factor empirically determined from wave equation analyses performed for six sites is given by:

$$F_p = 0.5\,(OCR)^{0.3}$$

The overconsolidation ratio (OCR) is estimated from: a) the measured undrained shear strength and Atterberg limit data, b) consolidation test results, and c) correlation with CPT tip resistance.

OCR is estimated from the measured undrained shear strength and Atterberg limit data using the following equations.

$$\frac{S_u}{S_{unc}} = (OCR)^{0.85}$$

where: S_u = actual undrained shear strength of clay having a given plasticity index (PI)
S_{unc} = undrained shear strength of the same clay if normally consolidated.

According to a relationship developed by Skempton (1944):

$$S_{unc} = \sigma_{vo}'(0.11 + 0.0037PI)$$

where: σ_{vo}' = effective overburden pressure
PI = plasticity index.

OCR may also be estimated from CPT tip resistances using the following equations:

$$\sigma_p' = 0.33\,(q_c - \sigma_{vo}')$$

$$OCR = \sigma_p' / \sigma_{vo}'$$

where: σ_{vo}' = effective overburden pressure
σ_p' = preconsolidation stress
q_c = cone tip resistance

4.1.2 Wave Equation Analyses

Wave equation analysis of pile driving is based on the discrete element idealization of the hammer-pile-soil system formulated by Smith (1960). The parameters used in the wave equation analysis can be divided into three groups: 1) hammer parameters, 2) pile parameters, and 3) soil parameters. These parameters are discussed in the following paragraphs.

Hammer Parameters. Air/steam and diesel hammers are modeled by four segments: 1) the ram as a weight with finite stiffness, 2) the hammer cushion as a weightless spring with finite stiffness, 3) the pile cap (helmet) as a weight with infinite stiffness, and 4) a pile cushion as a spring with finite stiffness. For hydraulic hammers, a hammer cushion is not used; the ram impacts directly on the pilecap. In addition, pile cushions are not typically used when driving steel pipe piles.

The pile driving hammer is described by the:

- Rated hammer energy,
- Efficiency of the hammer,
- Weight of the ram,
- Weight of the pilecap,
- Hammer cushion stiffness and coefficient of restitution, and
- Pile cushion stiffness and coefficient of restitution (concrete piles).

The rated energy and the weight of the ram and pilecap are obtained from the manufacturer. The hammer efficiency and cushion properties are either the measured driving system performance data (estimated from Fugro's database for 23 offshore hammers and 12 cushion configurations) or published values.

For the analyses presented here, the Delmag D62 diesel hammer was used with a fuel setting of 2 while driving through the Young Bay Mud, and a maximum setting of 4 thereafter.

Pile Parameters. The pile is divided into an appropriate number of segments of approximately equal length. Each pile segment is modeled as a weight and a spring. The pile parameters consist of the diameter, the wall thickness schedule, modulus of elasticity of the pile material, unit weight of the pile material, free-standing length of pile (stick-up), and penetration below the bay floor. For these illustrative calculations, a total stick-up of 14.8 meters above the bay floor has been assumed for all piles. The required pile lengths below bay floor were derived from the pile capacity analyses.

A practical consideration for the handling of the piles should be the slenderness ratio, which can be defined as the ratio of the pile length to its diameter. For the pile lengths chosen, the slenderness ratios range from 73 to 137. It has been assumed that all the piles are driven in one section, with no attachment of add-ons and without the use of followers. Because increasing pile lengths onsite is significantly more problematic for the precast concrete piles than for the steel pipe piles, the implications of overdrive should be studied carefully for the precast concrete piles. Due to the perceived handling problems associated with slender concrete piles, the smaller of the two concrete piles has been limited in length to 39.6 meters (including 14.8 meters above bay floor) for the purpose of the drivability assessment.

Soil Parameters. The soil resistance is distributed along the side of each embedded element and at the pile toe. During driving, the static component of resistance on each element is represented by an elastic spring with a friction block used to represent the ultimate static resistance. The dynamic component of resistance is modeled by a dashpot. There are essentially three soil parameters used in the wave equation analyses: 1) the quake (also referred to as the elastic ground compression) for the side and point of the pile, 2) the damping coefficient for the side and point of the pile, and 3) the percentage of the total resistance to driving at the pile toe.

It is recommended that the soil quake and damping parameters suggested by Roussel (1979) should be used in the wave equation analyses. These parameters were determined from a comprehensive correlation study performed for large-diameter offshore piles in which the driving records of 58 piles at 15 offshore sites in the Gulf of Mexico were analyzed. For the steel pipe piles, the side and point quakes are assumed equal, with a magnitude of 0.25 centimeter (cm) for stiff to hard clay, silt, and sand. For the precast concrete displacement piles, the side quake is also taken as 0.25 centimeter (cm), but the point quake is a function of the pile size. For the 0.41-meter concrete piles, a point quake of 0.34 cm was used; however, for the 0.61-meter concrete piles, a point quake of 0.51 cm was used. Side damping in clay decreases with increasing shear strength, which is in agreement with the laboratory test results of Coyle and Gibson (1970) and Heerema (1979). Point damping of 0.49 second per meter is recommended for firm to hard clay, silt, and sand.

4.2 EXAMPLE CALCULATIONS FOR PILE DRIVABILITY

Specimen wave equation analyses were performed using the hammer parameters tabulated below. For the steel pipe piles, a uniform wall thickness of 12.7 mm was assumed for the entire length of the pile. This will inevitably change if this type of pile is selected with a variable wall thickness schedule to compensate for potential high stresses induced in certain sections of the pile.

Hammer Type	Hammer Efficiency (%)	Coefficient of Restitution, Ram / Pilecap	Coefficient of Restitution, Pilecap / Pile
Delmag D62	80	0.80	0.50

4.2.1 Computed Soil Resistance to Driving

The soil resistances to driving (SRD) were computed using the methods outlined in the preceding sections, and the results are summarized in Plates D1-B.1 through D1-B.4.

4.2.2 Pile Run

The term "pile run" is used to describe the penetration of the pile due to self weight and the weight of the hammer. Estimates of the pile run due to hammer placement should be made by comparing the combined weight of the first pile section (or the total weight if only one section is adopted) and the hammer with the calculated lower-bound soil resistance to driving. Estimates of pile run for the example cases analyzed are tabulated below along with the summary of drivability results.

4.2.3 Blow Counts

The Young Bay Mud (YBM) and Old Bay Mud/Upper Alameda Marine (OBM/UAM) sequences primarily comprise marine clay sediments. When driving steel pipe piles through those sediments, experience suggests that the coring cases are generally representative of conditions during continuous driving, while the plugged cases are representative of conditions subsequent to significant delays. For the solid piles, the 'plugged' case is applicable in all instances.

The initial pile section is expected to "run" to a few meters above the base of the YBM sediments under the weight of the pile and hammer. Blow counts within the YBM sediments are expected to be very low (under 5 blows per 0.25 meter).

The Merritt-Posey-San Antonio (MPSA) Formations lie beneath the Young Bay Mud and consists of dense sands with stiff clay layers. The dense sand layers within these formations cause a significant increase in predicted blow counts, the magnitude of which is governed by the thickness of the sand layers. Stevens et al. (1982) considered the upper- and lower-bound plugged case to be a reasonable prediction of the driving behavior in dense sands; therefore, it is recommended that these be used as an indication of driving behavior in this sequence. Although the blow counts in this sequence are predicted to reach 65 blows per 0.25 meter in the upper-bound case for the larger of the two concrete piles analyzed, this is not considered to cause installation problems for the Delmag D62 hammer. However, this conclusion is highly sensitive to the layer thickness (relative to the chosen pile diameter) and soil resistance parameters chosen for the MPSA. This stratum should be considered carefully during the detailed drivability analyses.

Within the Old Bay Mud/Upper Alameda Marine (OBM/UAM), the drivability analyses suggest that the piles considered can be driven relatively easily with the Delmag D62 hammer. The blow counts for all cases were generally less than 30 blows per 0.25 meter. These blow counts suggest that delays during driving in the clay sediments of the OBM/UAM sequence are unlikely to significantly impact the installation process when using the pile and hammer combination chosen for these example calculations.

4.2.4 Driving Stresses

Generally, the highest stress level in the life of the pile occurs during driving. For efficient utilization of both driving hammer and pile material, it is desirable to stress the pile up to its practical allowable driving stresses. The high strain rate and temporary nature of the loading allow a higher, more sustainable pile stress than for static loading. However, for precast-prestressed concrete piles, driving stresses need to be closely controlled to minimize the possibility of damage to the pile during driving.

Based on AASHTO (1994) codes, the following allowable driving stress limits are recommended:

a. Tension Stress

- *Precast Concrete Piles* - The maximum recommended driving tension stress is a combination of the net pre-stress (f_{pe}) and concrete compressive strength (f_c'). Assuming an f_{pe} of 8 megapascals (MPa) and f_c' of 42 MPa, the recommended allowable tension driving stress is approximately 10 MPa.
- *Steel Pipe Piles* - Tensile stress does not usually govern allowable driving stresses, but has been included for demonstration purposes in these analyses.

b. Compressive Stress

- *Precast Concrete Piles* - The recommended allowable compressive stress during driving is also a function of f_{pe} and f_c'. Assuming an f_{pe} of 8 MPa and f_c' of 42 MPa, the recommended allowable compression driving stress is approximately 28 MPa.
- *Steel Pipe Piles* - It is recommended that the maximum compressive stress in steel pipe piles be limited to 90 percent of the yield stress. For the purpose of these example evaluations, a yield stress of 340 MPa was assumed for steel pipe piles.

Data from the specimen drivability assessment indicate that the driving stresses were generally within the allowable values. To reduce the risk that the allowable driving stresses are not exceeded during the pile driving, it is recommended that the contractor monitors and controls the hammer fuel settings (diesel hammer), hammer stroke (hydraulic hammer), and pile cushion thickness (concrete piles) during driving.

4.3 RESULTS OF DRIVABILITY ANALYSES

Both the predicted blow count versus depth and the soil resistance to driving curves estimated from wave equation analyses for Temporary Tower Location B are illustrated in Appendix D on Plates D2-B.1 through D2-B.4. The summary tabulated below is of the estimated pile run and maximum predicted blow count and induced stresses predicted in the analyses.

NOT FOR DESIGN

Pile Type and Size	Pile Length[a] (meters)	Pile Run (meters)	Maximum Predicted Blow Count (blows/0.25 meter)	Maximum Compressive Stress (MPa)	Maximum Tensile Stress (MPa)
0.41-Meter-Diameter Steel Pipe Pile	55.6	14.0	24	240	55
0.61-Meter-Diameter Steel Pipe Pile	47.7	12.6	56	260	51
0.41-Meter-Square Precast Concrete Pile	39.6[b]	14.3	22	22	8
0.61-Meter-Square Precast Concrete Pile	44.4	11.8	65	14	8

[a] Includes 14.8-meter stick-up above mudline.
[b] Pile length limited for practical handling constraints. Total length required for axial capacity = 48.8 meters.

5.0 POST-INSTALLATION SETUP OF AXIAL CAPACITY

The process of driving piles into fine-grained soil results in a high degree of disturbance in the soil adjacent to the pile and the creation of excess pore water pressures in the surrounding soil. One consequence of this disturbance is a decrease in the as-driven capacity and a subsequent increase in the capacity with time as the excess pore pressures dissipate. The reduced as-driven pile capacities will need to be considered during the construction process. In general, it is recommended that the minimum factor of safety requirements specified above be followed at all times.

5.1 EVALUATION OF SETUP

On the basis of a large number of experiments in Gulf of Mexico clays with instrumented probes and piles having a wide variation in diameters and displacements, empirical design procedures have been developed that include the effects of both diameter and wall thickness on the rate of consolidation and setup (Bogard and Matlock, 1990). The recommended relationship describing the evolution of axial capacity with time is:

$$Q(t) = Q_u \left[0.2 + 0.8 \left(\frac{\dfrac{t}{t_{50}}}{1 + \dfrac{t}{t_{50}}} \right) \right]$$

where: $Q(t)$ = axial capacity at time, t
Q_u = ultimate static axial capacity
t_{50} = time required for dissipation of 50 percent of the excess pore pressures

Predictions of pile setup are dependent upon parameters such as soil permeability and the level of disturbance caused by the installation process. In view of the variability inherent with those parameters, predictions of pile setup are fraught with uncertainty. Therefore, although the above empirical relationship was developed as a lower bound of the available field measurements (i.e., the data that suggest slower rates of setup), consideration should be given to its use during design and scheduling.

5.2 EXAMPLE CALCULATIONS

From an examination of all available experimental data, the values of t_{50} for 0.41- to 0.61-meter-diameter steel pipe piles with a thin-wall cutting shoe were estimated to range from 2 to 7 days. The corresponding values of t_{50} for 0.41- to 0.61-meter precast concrete piles range from 12 to 40 days.

On the basis of the above relationship, the setup curves for side shear capacity for both 0.41- to 0.61-meter steel pipe and precast concrete piles are presented on Plates E1-B.1 and E1-B.2, respectively, in Appendix E. It can be seen from these two plates that the time required to reach approximately 90 percent of the ultimate skin friction capacity in clay layers can be expected to range from 0.5 to 2 months for the steel pipe piles, and from 2.5 to 9.5 months for the precast concrete piles. The time required to reach approximately 50 percent of the skin friction capacity may range from 1 to 2 weeks for the steel pipe piles and from 1 to 3 weeks for the precast concrete piles.

6.0 PILE INSTALLATION CONSIDERATIONS

In general, piling for temporary towers should be performed in accordance with the requirements of Section 49 of the Standard Specifications (Caltrans, 1995). However, since the design of temporary tower piling is to be performed by the contractor, it is recommended that a pile data table documenting design loads and required pile tip elevations be submitted to the engineer for approval as part of the working drawing submittal. Supplementary recommendations are presented as follows.

6.1 DRIVING SYSTEM SUBMITTAL

Prior to installing driven piling, the Contractor should provide a driving system submittal (that includes a drivability analysis) in conformance with the provisions in Section 5-1.02, "Plans and Working Drawings," of the Caltrans Standard Specifications (1995). All proposed driving systems (i.e., each hammer that may be brought onto the site) should be included in the submittal. It is recommended that a minimum of 3 weeks be provided exclusively for review of the driving system submittal.

The driving system submittal should contain an analysis showing that the proposed driving systems will install piling to the design tip elevation in accordance with the criteria described in the subsequent sections. Drivability analyses should be performed for each temporary tower location.

Drivability studies included in the submittal should be based on a wave equation analysis done by using a computer program that has been approved by the engineer. The analysis should be performed for the pile-schedule/details shown on the contractor's design drawings. Drivability studies should model the Contractor's proposed driving systems (including the driving shoe, hammer, capblock, and pile cushion) as well as determine driving resistance and pile stresses for assumed site conditions. The analyses should consider a range of total soil resistance to driving and associated percentage shaft resistance. For steel pipe piles, both plugged and unplugged cases should be considered. The range of soil resistance to driving and percentage shaft resistance should be determined for site conditions ranging from 5 meters above to 5 meters below the specified pile tip elevation shown on the plans. Separate analyses should be completed at elevations above the specified pile tip elevations (e.g., within the dense Merritt Sand layers) where difficult driving is anticipated. As a minimum, submittals should include the following:

1. Complete description of soil parameters used, including soil quake and damping coefficients, distribution of skin friction, percentage shaft resistance, and total soil resistance to driving

2. List of all hammer operation parameters assumed in the analysis, including rated energy, stroke limitations, and hammer efficiency
3. Completed "Pile and Driving Data Form"
4. Results from drivability analyses should include:
   a. Estimates of pile penetration due to self weight and the weight of the hammer
   b. Plots of maximum pile head and pile toe compressive stress versus blows per 250 mm
   c. Plots of maximum pile tensile stress versus blows per 250 mm; and
   d. Plots of soil resistance to driving versus blows per 250 mm
5. Copies of all test results from any previous pile load tests, dynamic monitoring, and all driving records used in the analyses.

6.2 PILE REFUSAL CRITERIA

The definition of pile refusal is primarily for contractual purposes to define the point where pile driving with a particular hammer should be stopped and other methods instituted (e.g., jetting or using a larger hammer). The definition of pile refusal is also meant to reduce the possibility of causing damage to the pile and hammer.

Section 49-1.07 of the Caltrans Standard Specifications (1995) states that "When the blow count exceeds either 2 times the blow count required in 300 mm, or 3 times the blow count required in 75 mm for the design bearing load … additional aids shall be used to obtain the specified penetration." Note, however, that piles driven through dense sand layers of the Merritt-Posey-San Antonio Formations and designed to tip in the underlying clay layers will likely encounter relatively high blow counts (probably in excess of the refusal criteria specified in Caltrans [1995]) at pile penetrations above the design tip elevation. Therefore, it is recommended that the refusal criteria specified in Caltrans (1995) be modified as discussed below.

The following refusal criteria are based on the assumption of a properly operating hammer. If refusal occurs and the driving system performance is inadequate, the hammer or cushion should be changed before remedial measures are undertaken. These recommendations are intended to serve as guidelines for the establishment of refusal criteria. Final refusal criteria should be developed for the particular hammer system(s) selected and approved to drive the production piles.

Concrete piles are more susceptible to pile damage than steel piles and, therefore, typically have lower specified refusal criteria than large-diameter steel pipe piles. A preliminary recommended refusal blow count criteria for concrete piles is 125 blows per 0.25 meter, or 65 blows per 0.12 meter. For steel pipe piles, consideration can be given to the use of API

(1993a,b) refusal criteria. This refusal blow count should be re-evaluated for the particular hammer system selected to drive the production piles.

If a pile reaches refusal short of design penetration, pile acceptance should be evaluated by the engineer before remedial installation procedures are undertaken. When techniques other than driving are used to advance the pile, conditions assumed in the computation of ultimate pile capacity based on driving alone may not be met, and pile capacities may have to be recomputed to more closely reflect the actual installation procedure.

6.3 DELAYS AND REDRIVING

During driving, it may be necessary to interrupt driving operations to change hammers or perhaps add pile sections for pipe piles. The interruption to driving operations may last 12 to 24 hours. Delays on the order of several days may result from bad weather or equipment breakdown. The required welding time for add-ons increases with wall thickness. During this time, many clays (and some fine-grained sands) will cause setup as excess pore pressures dissipate and the soil particles reorient themselves. Upon redriving piles after setup has occurred, increased blow counts may be experienced for delays longer than about 2 hours. To reduce the potential for encountering refusal above the design tip elevations, interruptions in driving should be kept as short as possible. Worn cushions should be replaced and back-up hammers should be available. If delays during driving are foreseeable, driving should not be stopped when the pile tip is within 5 pile diameters of a sand layer shown on the typical soil profiles.

6.4 ALLOWABLE DRIVING STRESS CRITERIA

Generally, the highest stress level in the life of a pile occurs during driving. For efficient utilization of both the pile driving hammer and pile material, it is desirable to stress the pile to the practical limit during driving. The high strain rate and temporary nature of the loading allow a substantially higher allowable stress than for static loading.

For steel piles, it is recommended that the allowable driving stress be limited to 90 percent of the yield stress of steel.

For precast-prestressed concrete piles, the following allowable driving stress limits are recommended:

- **Tension Stress.** The maximum recommended driving tension stress is the net pre-stress (f_{pe}) plus three times the square root of the concrete compressive strength (f_c').
- **Compressive Stress.** The maximum recommended allowable compressive stress is $(0.85)f_c' - f_{pe}$.

The contractor's driving system submittal should document procedures to minimize the potential for the above allowable stresses being exceeded. Additionally, the wave equation analyses should indicate that the allowable stresses are not exceeded for the range of anticipated soil resistances to driving.

6.5 MINIMUM BEARING CRITERION

A minimum bearing criterion is specified in Section 49-1.08 of the Caltrans Standard Specifications (1995). In typical Caltrans practices, if the piles do not reach sufficient capacity at the specified tip elevation, the pile will need to be driven further such that the piles achieve the specified minimum bearing. According to Caltrans (1995), bearing values for driven piles are evaluated using the following formula:

$$P = \frac{E_r}{6\left(s + 2.54\right)}$$

where: P = safe load in kilonewtons
E_r = manufacturer's rating for joules of energy developed by the hammer
s = penetration per blow in millimeters, averaged over the last few blow counts

As described previously, note that piles driven through primarily clay soils will experience setup after the end of driving. Therefore, a significant amount of time may be required for soil resistance to increase to a point where the required minimum bearing criterion is met. To reduce the potential for excessive overdrive allowances and/or the need for pile splicing, it is recommended that a minimum of two pile restrikes be performed for at least 25 percent of the piles at each tower location at positions to be selected by the engineer to evaluate if piles have the required design capacity.

The timing for pile restrikes if minimum bearing criteria are not met at the design tip elevation should be evaluated once the pile type and length have been selected. Preliminarily, it is recommended that a minimum of 7 days be allowed between the end of driving and the first pile restrike, and between pile restrikes.

6.6 PILE RESTRIKES

During pile restrikes, the pile should be advanced a minimum of 75 mm, and the restrike blow count should be taken as the average blow count over the first 50 mm of driving. During the restrike, the contractor's approved hammer should be operating at the manufacturer's rated energy. For example, diesel hammers should be operated at the maximum fuel setting and hydraulic hammers should be operated at the maximum stroke. Diesel hammers should be warmed up prior to restriking the pile by having driven another pile with at least 100 blows. If pile cushions are used, restrikes should be conducted with a used 180- to 250-mm thick plywood pile cushion to ensure that sufficient energy is transferred to the pile during the restrike. It is recommended that a used pile cushion be defined as having received at least 100 blows from an acceptable pile driving hammer at the maximum rated energy setting.

6.7 PILE ACCEPTANCE CRITERIA

Preliminary pile acceptance criteria are specified here to assist with the development of a 100% Design Submittal. The recommendations presented here should be reviewed and refined once the final design alternative is selected.

It is recommended that a pile from a particular pier be considered acceptable if: 1) the pile has been driven to the design tip elevation, and 2) if at least 25 percent of the piles at that pier location meet the minimum bearing criteria. In lieu of blow count acceptance, the piles may be evaluated based on Pile Driving Analyzer (PDA) measurements and Case Pile Wave Analysis Program (CAPWAP) analyses.

If minimum blow count criteria are not met at the specified tip elevation after two pile restrikes, the piles should be driven a minimum of 1.0 meter and then evaluated for minimum bearing criteria. It is recommended that mechanical splices not be allowed at the top of the pile. The contractor should provide an appropriate overdrive allowance in the event that minimum blow count criteria are not met at the required pile tip elevation.

If a pile reaches refusal short of design penetration, pile acceptance should be evaluated by a geotechnical engineer before remedial installation procedures or design modifications are undertaken. When techniques other than driving are used to advance the pile, conditions assumed in the computation of ultimate pile capacity based on driving alone may not be met, and pile capacities may have to be recomputed to more closely reflect the actual installation procedure.

It is recommended that piles driven to refusal with a satisfactorily performing hammer approved by the engineer to within 3 meters of the design penetration be accepted. Piles driven to refusal above design penetration can also be accepted if dynamic monitoring and CAPWAP analyses indicate that the required compressive and tensile capacities are mobilized. In cases where refusal is the result of unsatisfactory hammer performance, the problem should be corrected and the pile redriven.

6.8 SUPPLEMENTARY INSTALLATION TECHNIQUES IN THE EVENT OF PILE REFUSAL

The most economical pile installation procedure is by driving alone without resorting to supplemental procedures. The computed ultimate capacity of driven pipe piles presented is based on the assumption that the piles will be driven to the desired penetration without supplemental drilling or jetting. When techniques other than driving are used to aid pile installation, conditions assumed in computations based on driving alone may not be met. In this case, computed capacities must frequently be adjusted to fit actual installation conditions.

Supplementary pile installation procedures that may be considered under various circumstances, including the possible effects that these procedures may have on pile capacity, have been presented by Sullivan and Ehlers (1972). Application of these or other procedures to aid ordinary driving requires field decisions that take into account many factors beyond the scope of this report.

It is recommended that, if used, supplementary procedures be chosen and applied under close engineering supervision. These procedures should be selected considering both construction expediency and their effects on pile capacity. It is recommended that an engineer who is thoroughly familiar with the effects that supplemental pile installation methods have on the parameters used to determine pile capacity be present during construction. Also, since supplemental procedures may be required, it is recommended that the proper equipment necessary to achieve the desired penetration be available at the site when platform installation is started. This should avoid costly delays.

7.0 OBSERVATIONS FROM THE PILE INSTALLATION DEMONSTRATION PROJECT

A Pile Installation Demonstration Project (PIDP) was performed in the project area between October and December 2000. During that program, three 2.4-meter-diameter steel pipe piles were driven to penetrations of approximately 93 meters. A description of the program and a documentation of the results are presented in the Pile Installation Demonstration Project geotechnical report (Fugro-EM, 2001b). Although the temporary tower piles are anticipated to be substantially shorter and of smaller diameter, the results of the PIDP program provide insight into the behavior of driven piles in Bay Mud.

7.1 SOIL RESISTANCE TO DRIVING

Analyses of dynamic monitoring data from that program was used to generate approximate estimates of soil resistance to driving. Preliminary comparisons of those data to the predictions of the methods described in this report were generally satisfactory. On average, the interpreted soil resistances to continuous driving at shallow penetrations were somewhat lower than predicted using the Stevens et al. (1982) method. However, that observation may be related to the marginal suitability of dynamic monitoring data when using high energy hammers to advance large-diameter piles in relatively soft soil.

7.2 PILE SETUP

Approximate predictions of the rate at which soil setup was occurring within the clay layers penetrated by the PIDP piles are reported in Fugro-EM (2001b). In general, the data tend to suggest that setup around the large-diameter piles occurred somewhat faster than was predicted using the Bogard and Matlock (1990) procedure described in Section 5.0. That observation is consistent with the variations between conditions at the PIDP project area and at the areas from which the data used to develop the Bogard and Matlock procedure were derived. In general the PIDP piles involve: 1) lower pile displacement ratios, 2) slightly more plastic soil, and 3) foundation support from a greater number of overconsolidated crustal zones. The dynamic monitoring data collected during pile restrikes suggest that pile setup occurs more rapidly within those crustal zones.

7.3 ULTIMATE PILE CAPACITY

Evaluations of pile capacity were made from the pile monitoring data collected during several pile restrikes. Those evaluations suggested that the skin friction capacity of the pile was approaching that predicted using the API (1993a,b) method and was likely to surpass those estimates. That observation indicates that the use of the API (1993a,b) procedure is likely appropriate for the design of these temporary tower foundations.

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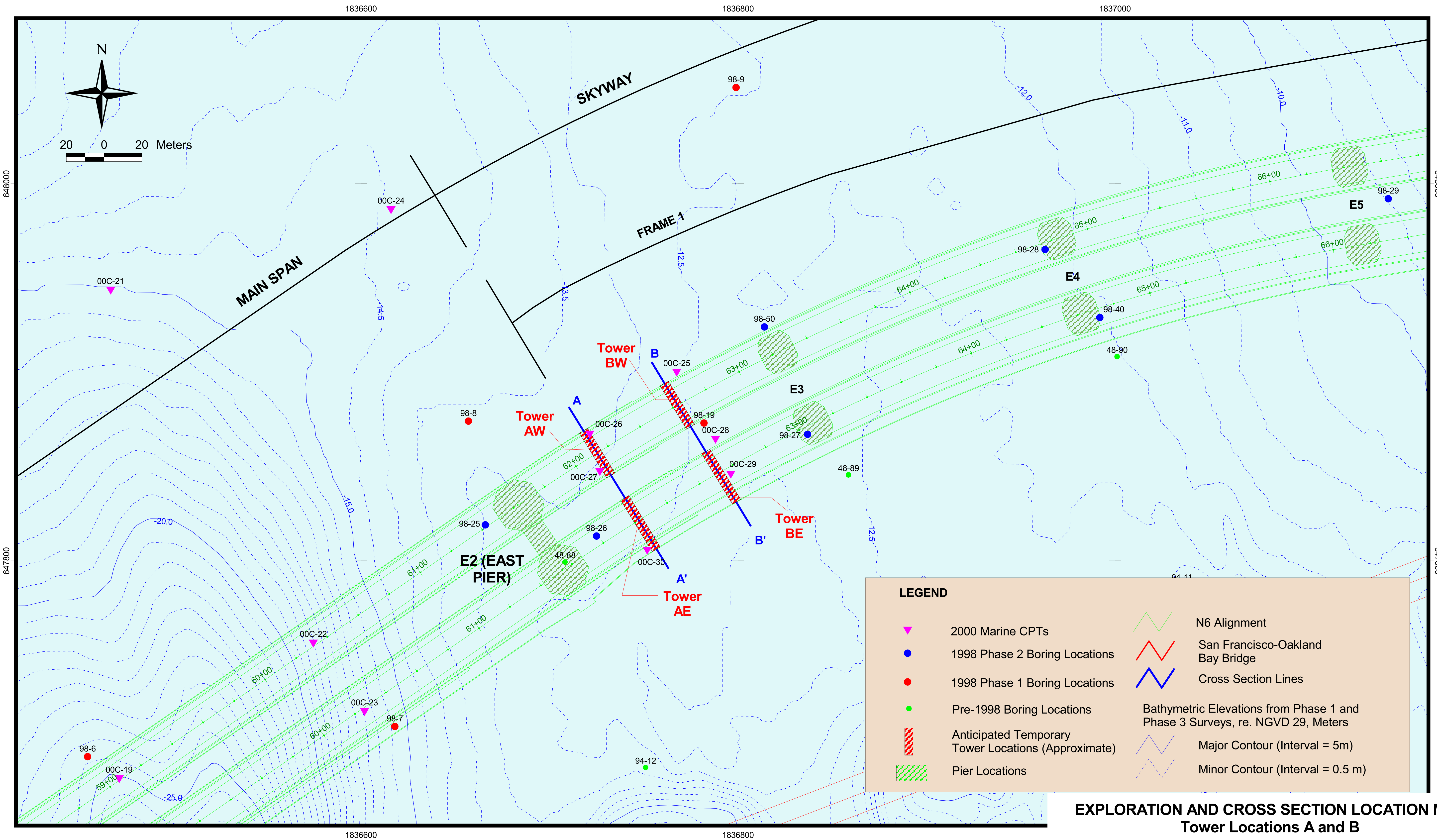
PLATES



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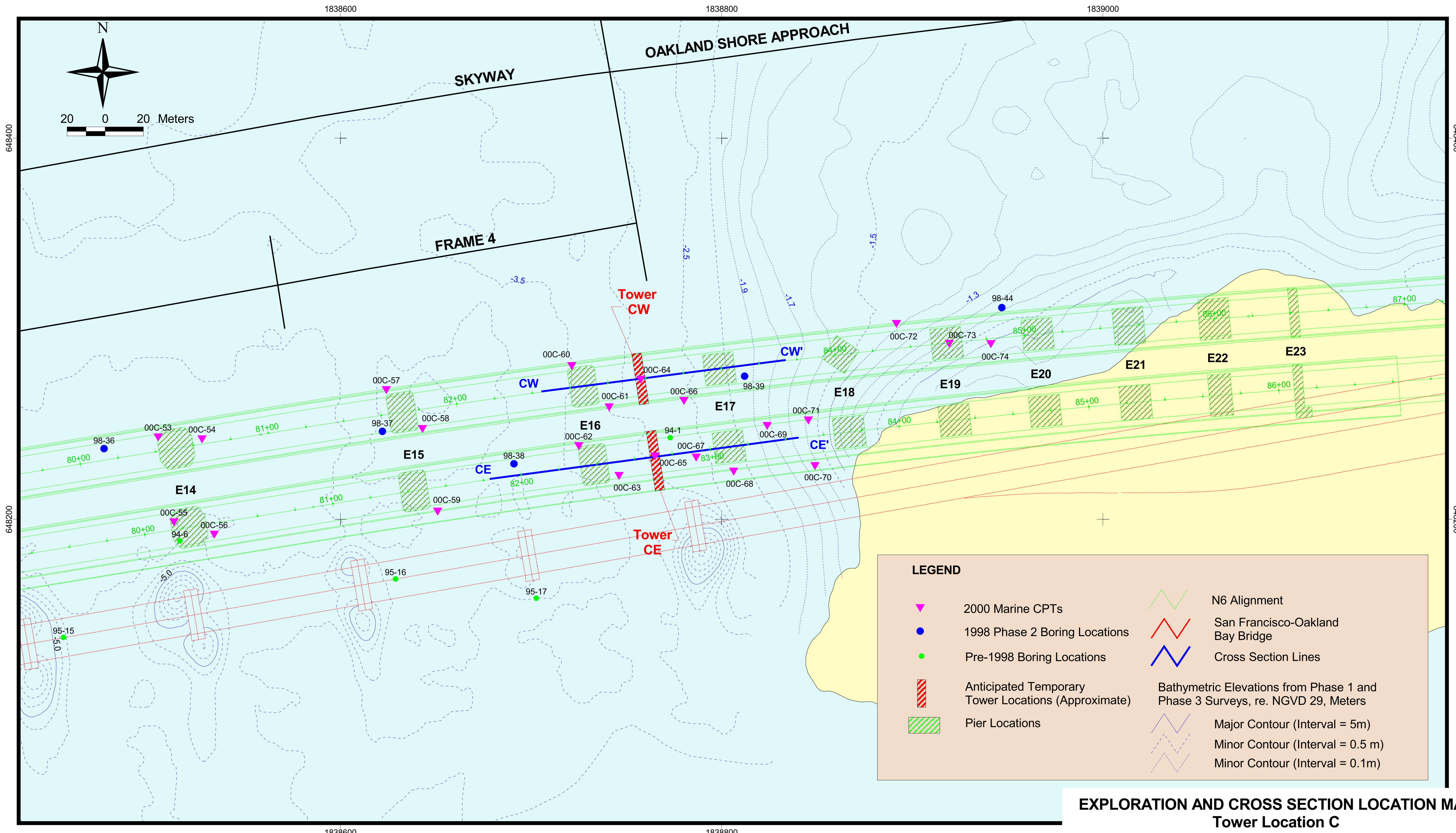
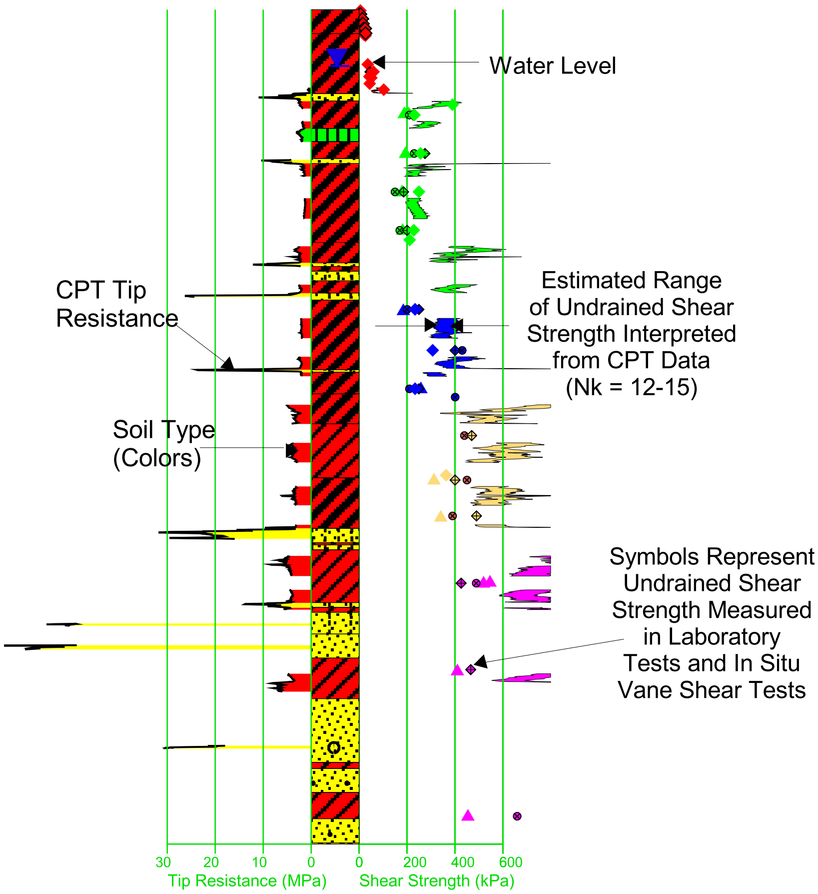
EXPLORATION AND CROSS SECTION LOCATION MAP
Tower Locations A and B
SFOBB East Span Seismic Safety Project

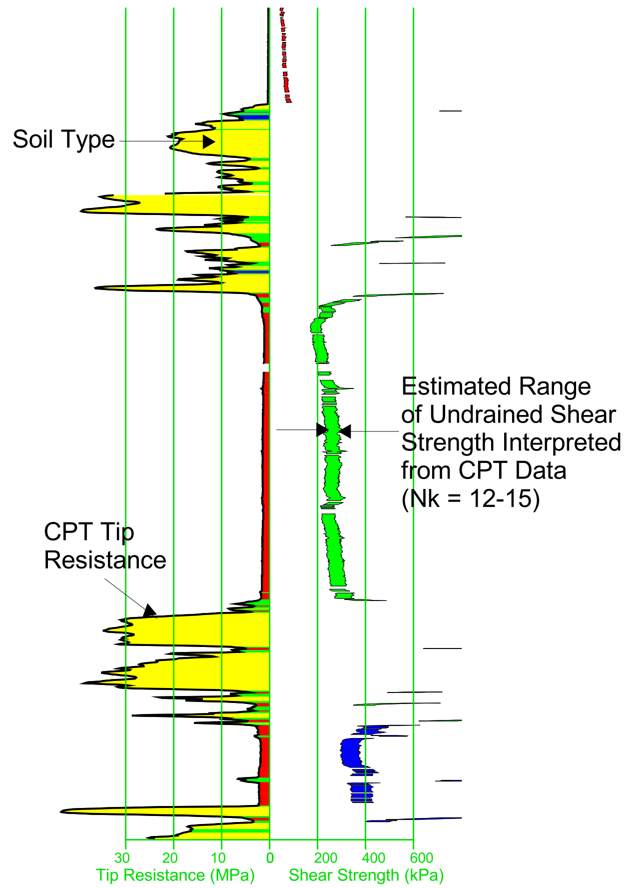


EXPLORATION AND CROSS SECTION LOCATION MAP
Tower Location C
SFOBB East Span Seismic Safety Project

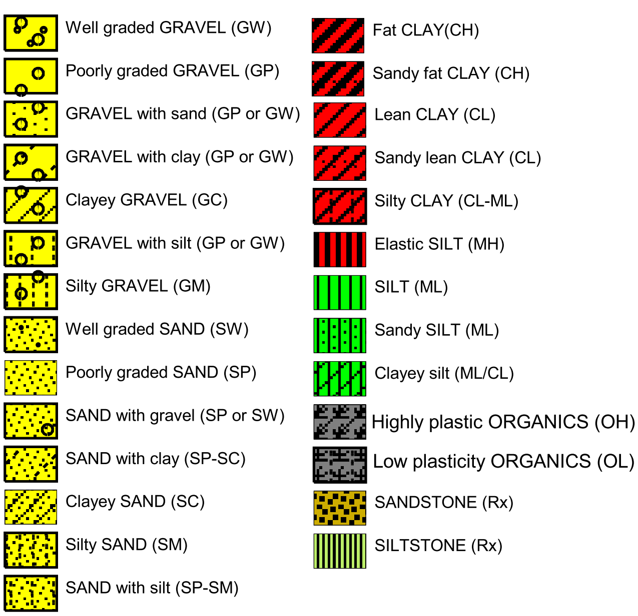
SOIL BORING LITHOLOGY WITH UNDRAINED SHEAR STRENGTH



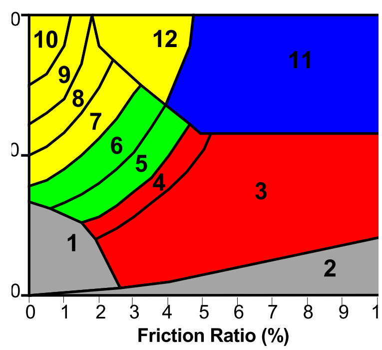
CPT SOUNDING WITH UNDRAINED SHEAR STRENGTH



KEY TO SOIL LITHOLOGY:



CPT CORRELATION CHART
(Robertson and Campanella, 1988)



Zone	Soil Behavior Type	U.S.C.S.
1	Sensitive Fine-grained	OL-CH
2	Organic Material	OL-OH
3	Clay	CH
4	Silty Clay to Clay	CL-CH
5	Clayey Silt to Silty Clay	MH-CL
6	Sandy Silt to Clayey Silt	ML-MH
7	Silty Sand to Sandy Silt	SM-ML
8	Sand to Silty Sand	SM-SP
9	Sand	SW-SP
10	Gravelly Sand to Sand	SW-GW
11	Very Stiff Fine-grained *	CH-CL
12	Sand to Clayey Sand *	SC-SM

*overconsolidated or cemented

Soil lithologic classifications based on the Robertson and Campanella (1988) soil behavior chart are sometimes inaccurate. For example, CPT data from many of the stiff to hard clay layers of the Old Bay Mud/Upper Alameda Marine sediments plot in soil behavior zones that correspond to silts and are shown in green on the cross sections.

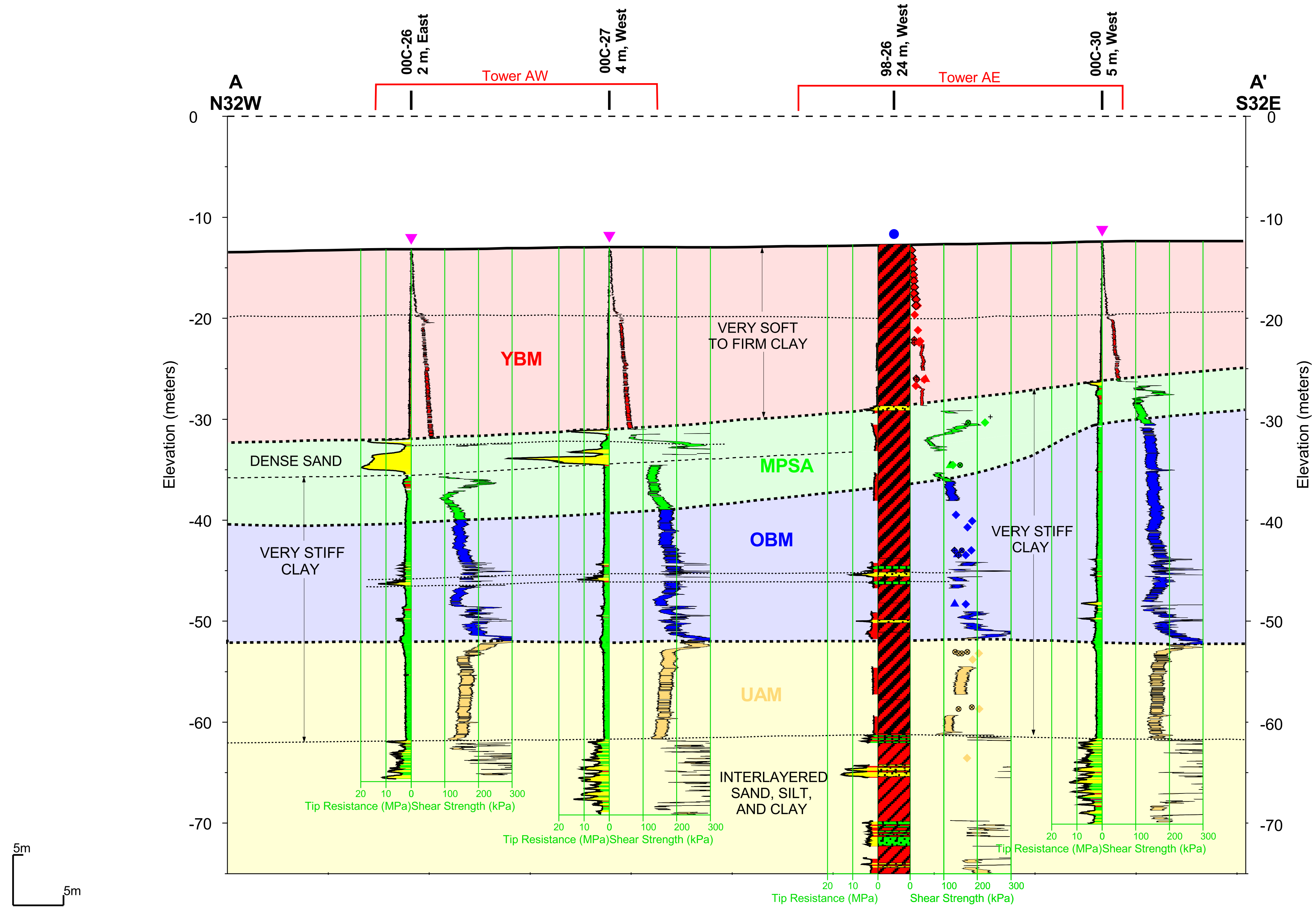
KEY TO BORING LOGS AND CPT SOUNDINGS ON CROSS SECTIONS

SFOBB East Span Seismic Safety Project

PLATE 3

Earth Mechanics

FUGRO



GENERAL NOTES:

1) Stratigraphic contacts are approximate and are interpreted from CPT soundings, borings, and seismic reflection survey data. Conditions vary both along and perpendicular to the section line.

2) Refer to Key to Cross Sections for descriptions of boring and CPT data shown above.

3) Lithology for pre-1998 borings are in some instances modified from those shown on the Caltrans Log of Test Boring sheets. Modifications were based on subsequent laboratory test results and extrapolation from adjacent 1998 borings.

SHEAR STRENGTH SYMBOLS

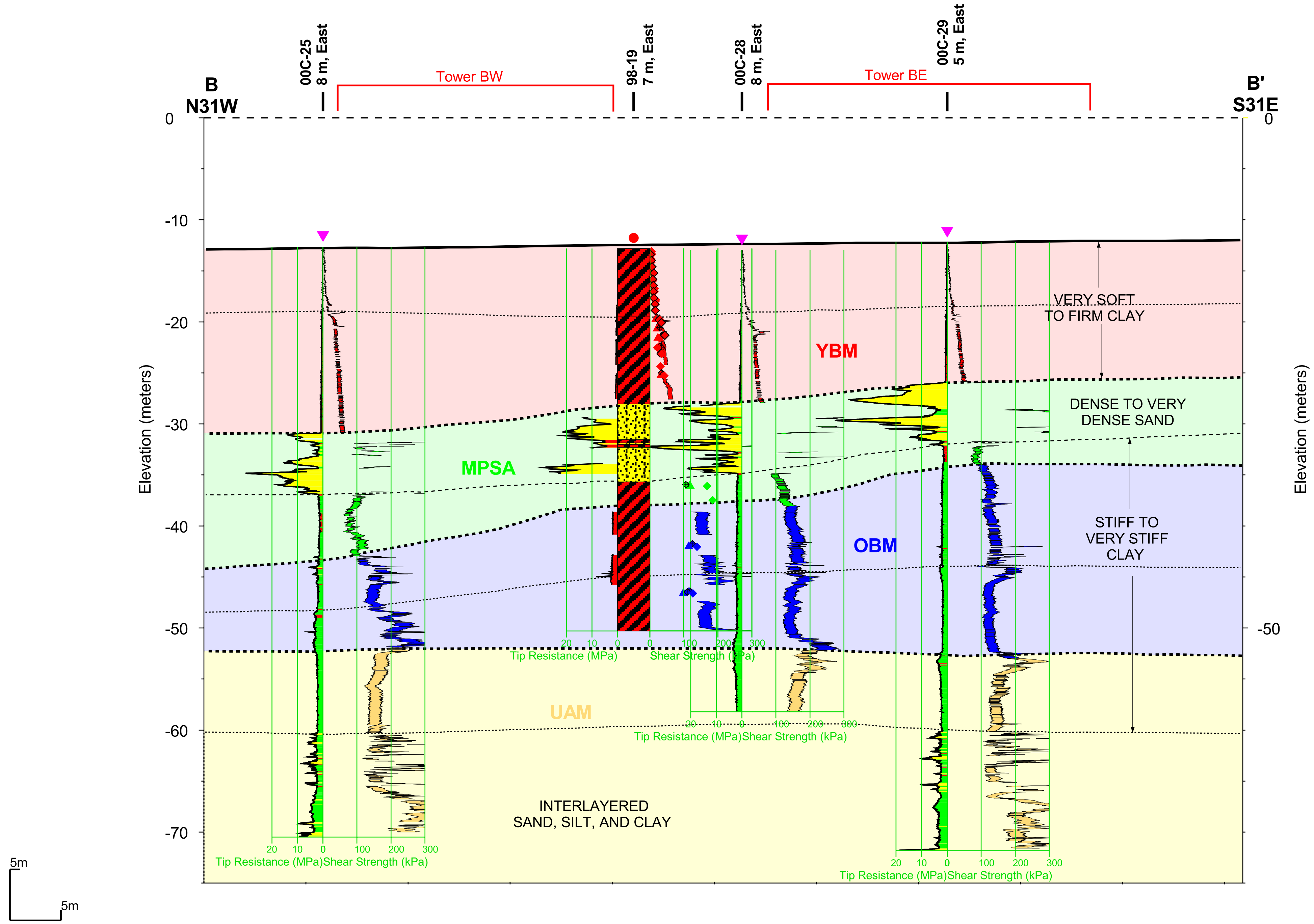
- ▲ Unconsolidated Undrained (UU)
- □ Unconfined Compression (UC)
- ⊗ Pocket Penetrometer (PP)
- Torvane (TV)
- ◆ Miniature Vane (MV)
- Remote Vane (RV)
- \+ Strength Exceeds Capacity of Measuring Device

KEY TO GEOLOGIC UNITS

YBM	Young Bay Mud
MPSA	Merritt-Posey-San Antonio Formations
OBM	Old Bay Mud
UAM	Upper Alameda Marine

SUBSURFACE CROSS SECTION A-A'
With Undrained Shear Strength
SFOBB East Span Seismic Safety Project

PLATE 4



GENERAL NOTES:

1) Stratigraphic contacts are approximate and are interpreted from CPT soundings, borings, and seismic reflection survey data. Conditions vary both along and perpendicular to the section line.

2) Refer to Key to Cross Sections for descriptions of boring and CPT data shown above.

3) Lithology for pre-1998 borings are in some instances modified from those shown on the Caltrans Log of Test Boring sheets. Modifications were based on subsequent laboratory test results and extrapolation from adjacent 1998 borings.

SHEAR STRENGTH SYMBOLS

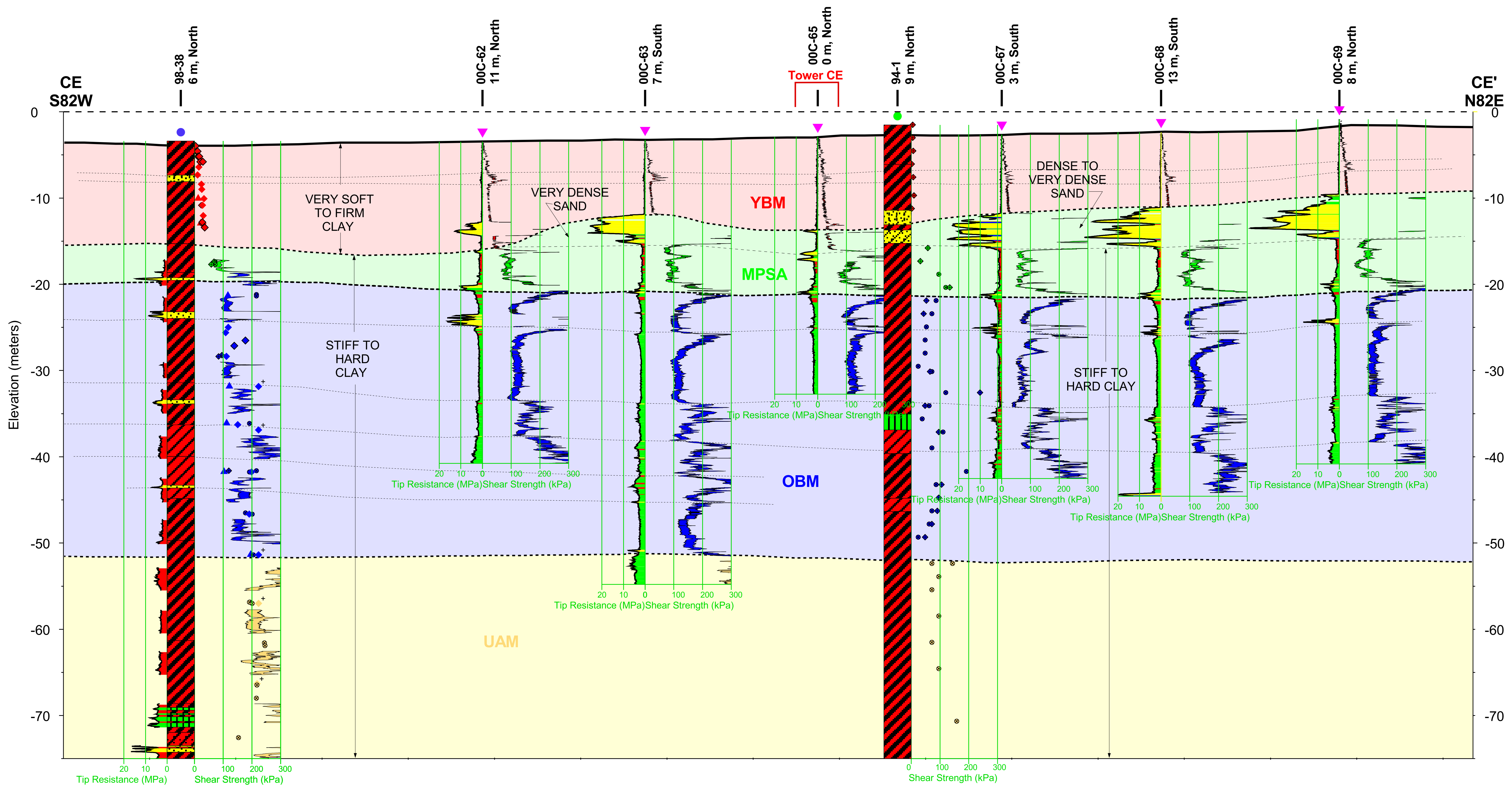
▲	Unconsolidated Undrained (UU)
□	Unconfined Compression (UC)
⊗	Pocket Penetrometer (PP)
◈	Torvane (TV)
◆	Miniature Vane (MV)
◇	Remote Vane (RV)
+	Strength Exceeds Capacity of Measuring Device

KEY TO GEOLOGIC UNITS

YBM	Young Bay Mud
MPSA	Merritt-Posey-San Antonio Formations
OBM	Old Bay Mud
UAM	Upper Alameda Marine

SUBSURFACE CROSS SECTION B-B'
With Undrained Shear Strength
SFOBB East Span Seismic Safety Project

PLATE 5



GENERAL NOTES:

1) Stratigraphic contacts are approximate and are interpreted from CPT soundings, borings, and seismic reflection survey data. Conditions vary both along and perpendicular to the section line.

2) Refer to Key to Cross Sections for descriptions of boring and CPT data shown above.

3) Lithology for pre-1998 borings are in some instances modified from those shown on the Caltrans Log of Test Boring sheets. Modifications were based on subsequent laboratory test results and extrapolation from adjacent 1998 borings.

5m
5m

SHEAR STRENGTH SYMBOLS

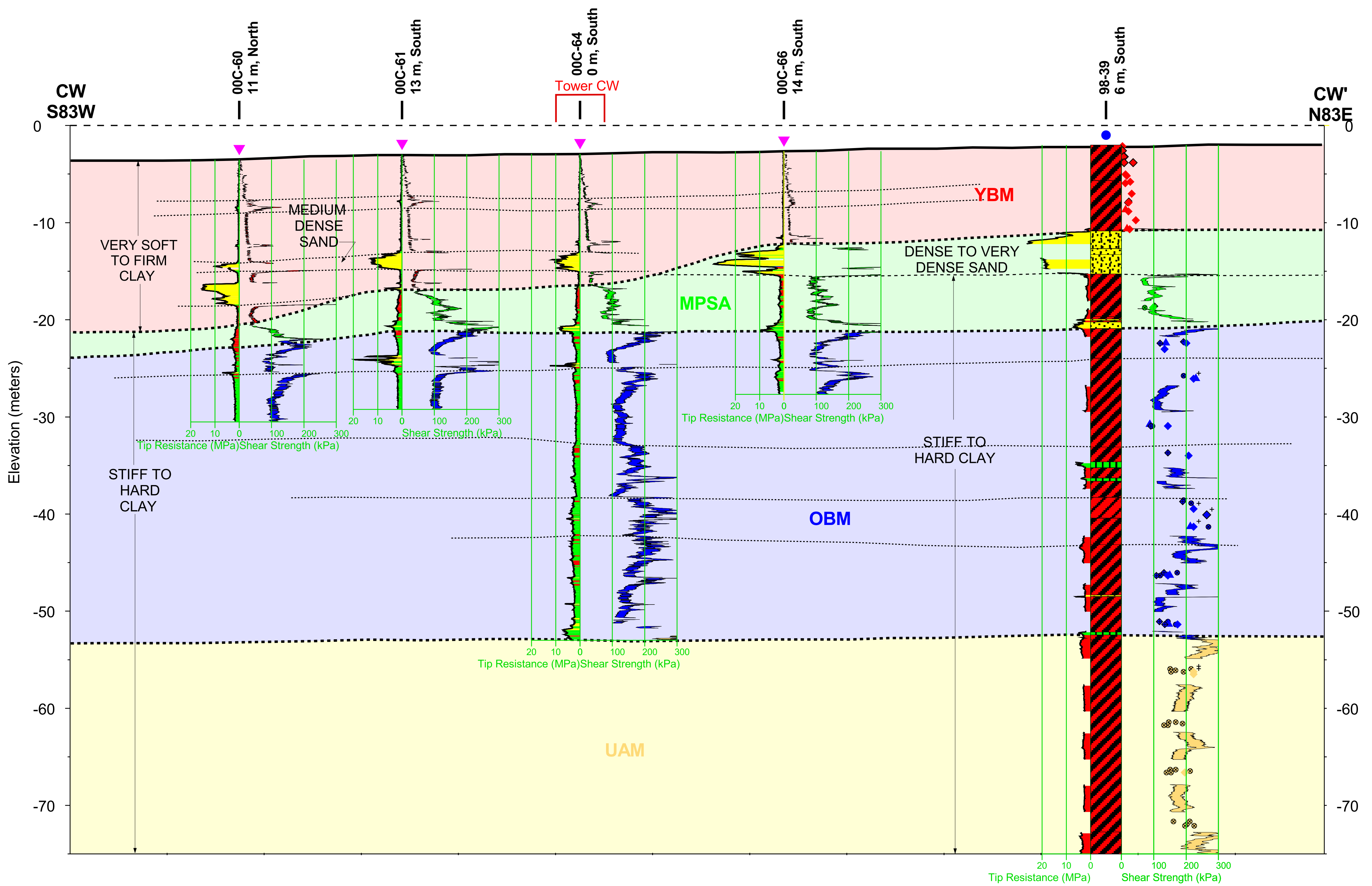
- ▲ Unconsolidated Undrained (UU)
- □ Unconfined Compression (UC)
- ⊗ Pocket Penetrometer (PP)
- ⊕ Torvane (TV)
- ◆ Miniature Vane (MV)
- ◇ Remote Vane (RV)
- \+ Strength Exceeds Capacity of Measuring Device

KEY TO GEOLOGIC UNITS

YBM	Young Bay Mud
MPSA	Merritt-Posey-San Antonio Formations
OBM	Old Bay Mud
UAM	Upper Alameda Marine

SUBSURFACE CROSS SECTION CE-CE'
With Undrained Shear Strength
SFOBB East Span Seismic Safety Project

PLATE 6



5m
5m

GENERAL NOTES:

1) Stratigraphic contacts are approximate and are interpreted from CPT soundings, borings, and seismic reflection survey data. Conditions vary both along and perpendicular to the section line.

2) Refer to Key to Cross Sections for descriptions of boring and CPT data shown above.

3) Lithology for pre-1998 borings are in some instances modified from those shown on the Caltrans Log of Test Boring sheets. Modifications were made based on subsequent laboratory test results and extrapolation from adjacent 1998 borings.

SHEAR STRENGTH SYMBOLS

- ▲ Unconsolidated Undrained (UU)
- □ Unconfined Compression (UC)
- ⊗ Pocket Penetrometer (PP)
- ⊕ Torvane (TV)
- ◆ Miniature Vane (MV)
- ◇ Remote Vane (RV)
- + Strength Exceeds Capacity of Measuring Device

KEY TO GEOLOGIC UNITS

YBM	Young Bay Mud
MPSA	Merritt-Posey-San Antonio Formations
OBM	Old Bay Mud
UAM	Upper Alameda Marine

SUBSURFACE CROSS SECTION CW-CW'
With Undrained Shear Strength
SFOBB East Span Seismic Safety Project

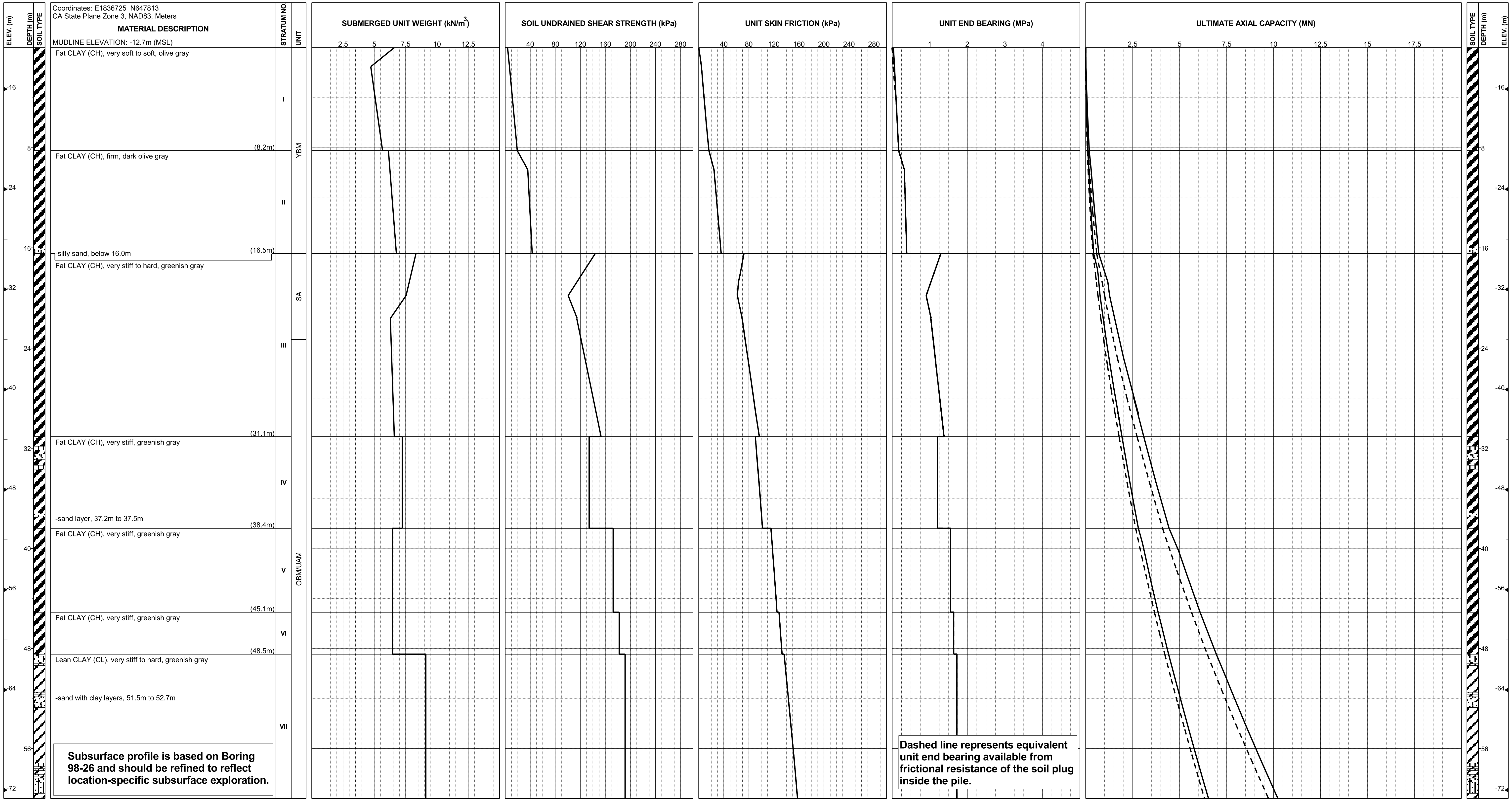
PLATE 7

APPENDIX A
STATIC AXIAL PILE CAPACITY ANALYSIS

APPENDIX A

EXAMPLE CALCULATIONS
NOT FOR DESIGN

Earth Mechanics
FUGRO



PRELIMINARY AXIAL PILE DESIGN PARAMETERS AND EXAMPLE RESULTS

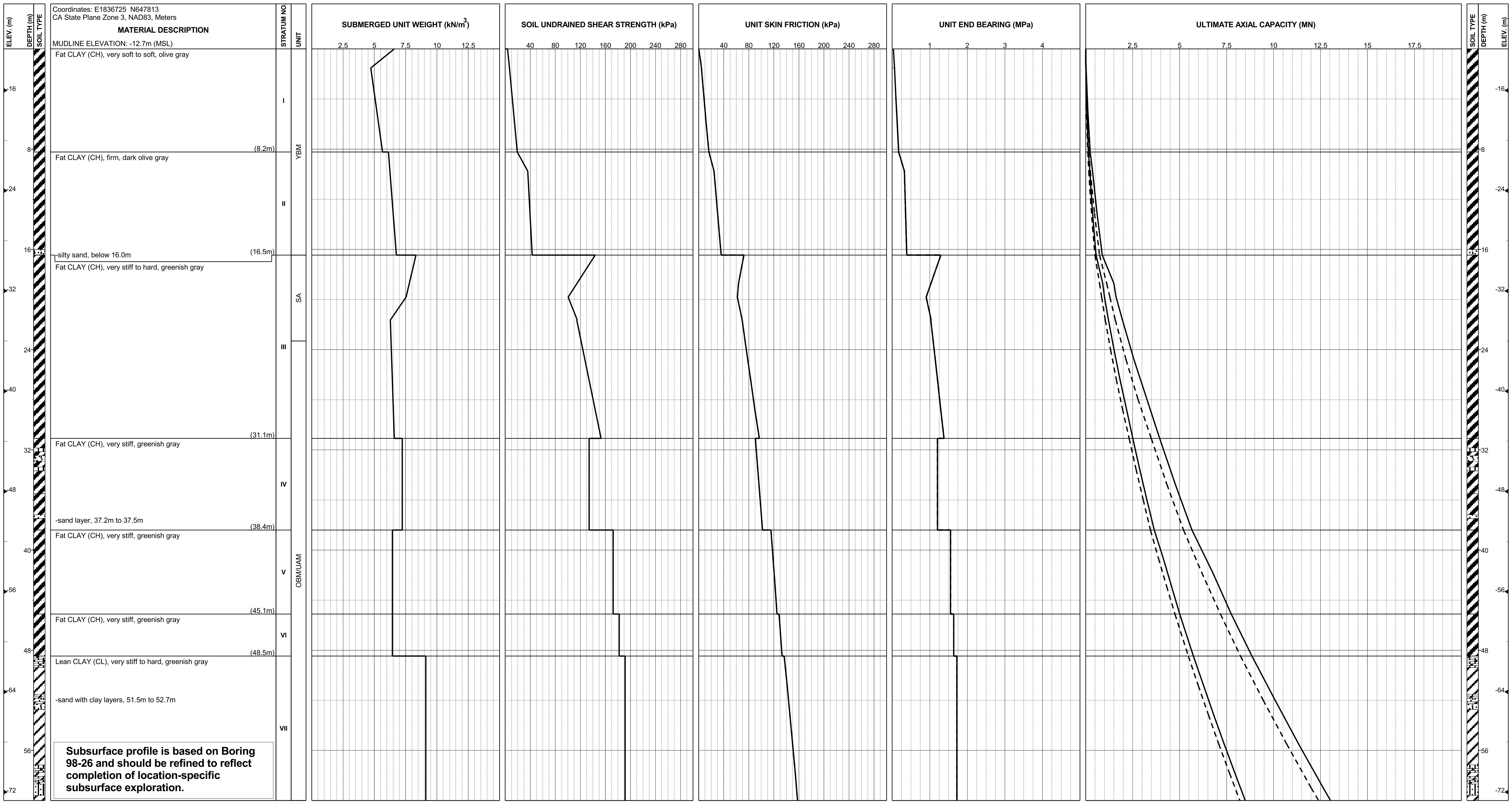
Skyway Temporary Tower: AE and AW, Pier E2 to E3

0.41- and 0.61-Meter-Diameter Steel Pipe Piles

SFOBB East Span Seismic Safety Project

EXAMPLE CALCULATIONS
NOT FOR DESIGN

Earth Mechanics FUGRO



PRELIMINARY AXIAL PILE DESIGN PARAMETERS AND EXAMPLE RESULTS
Skyway Temporary Tower: AE and AW, Pier E2 to E3
0.41- and 0.61-Meter-Square Precast Concrete Piles
SFOBB East Span Seismic Safety Project

EXAMPLE CALCULATIONS
NOT FOR DESIGN

Earth Mechanics
FUGRO

PRELIMINARY AXIAL PILE DESIGN PARAMETERS AND EXAMPLE RESULTS

Skyway Temporary Tower: AE and AW, Pier E2 to E3

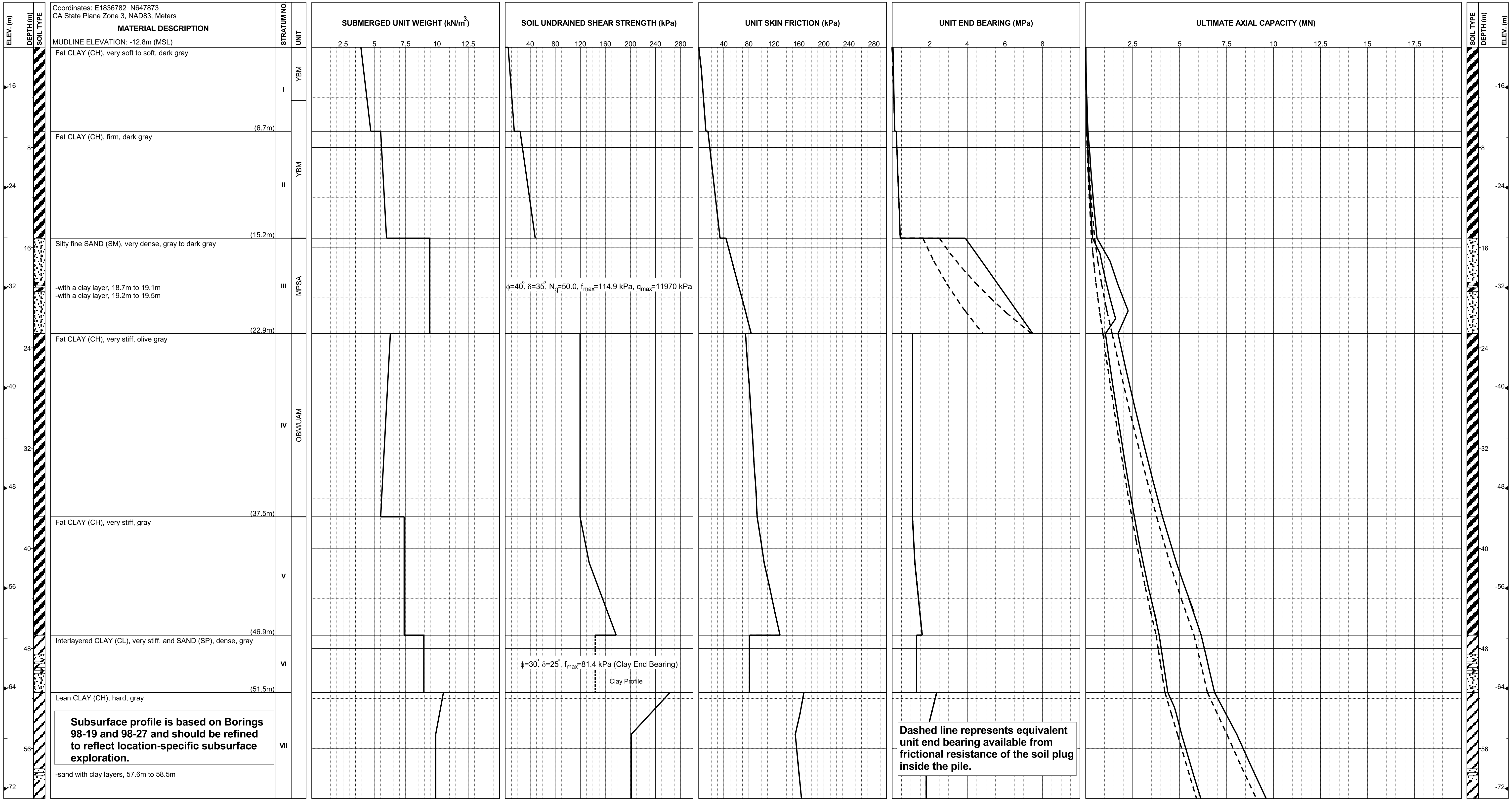
0.36-Meter Steel H Pile

SFOBB East Span Seismic Safety Project

SFOBB Task Order No. 5
Project No. 98-42-0054

EXAMPLE CALCULATIONS
NOT FOR DESIGN

Earth Mechanics
FUGRO



PRELIMINARY AXIAL PILE DESIGN PARAMETERS AND EXAMPLE RESULTS

Skyway Temporary Tower: BE and BW, Pier E2 to E3

0.41- and 0.61-Meter-Diameter Steel Pipe Piles

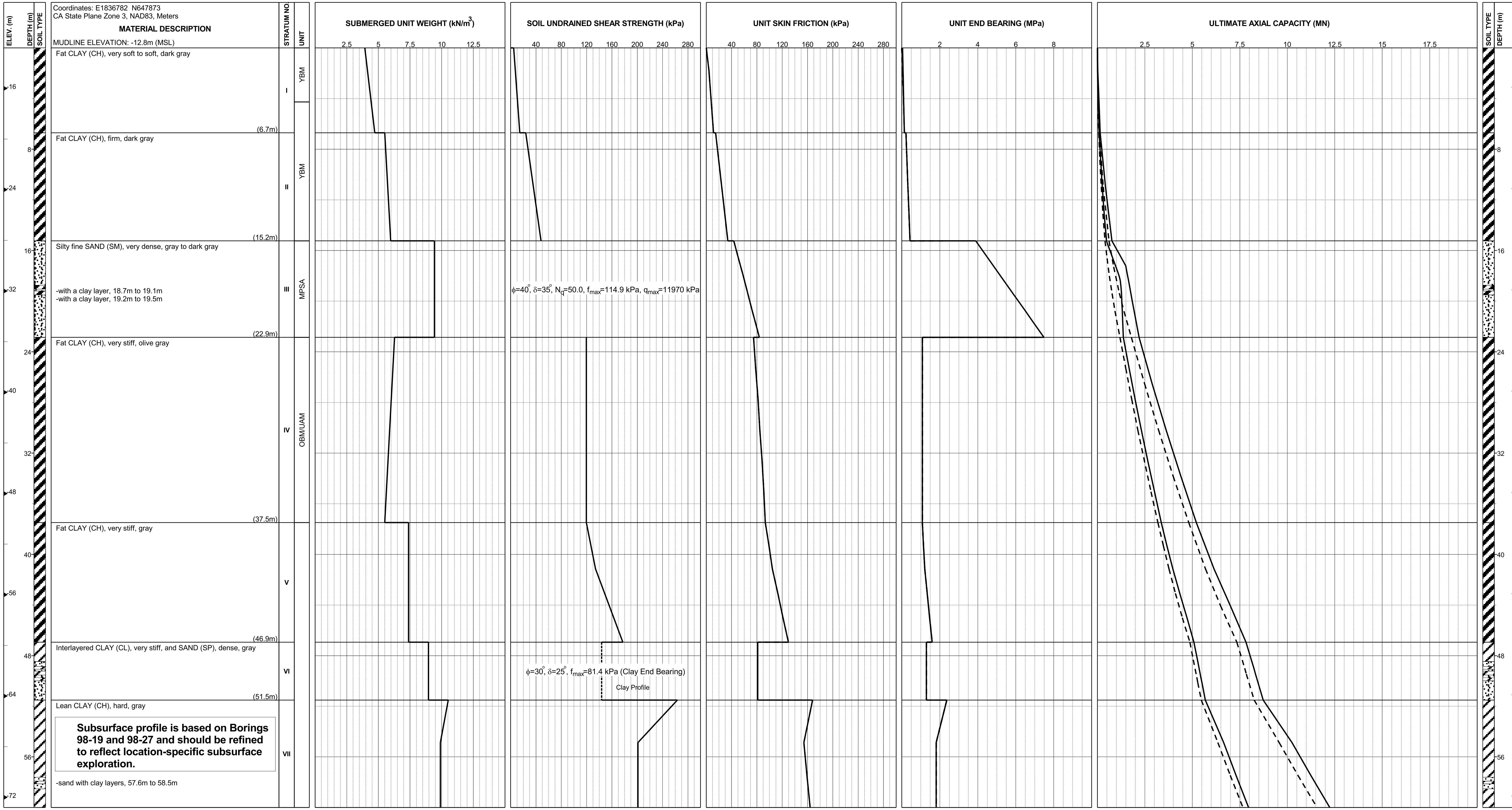
SFOBB East Span Seismic Safety Project

PLATE A1-B.1

SFOBB Task Order No. 5
Project No. 98-42-0054

EXAMPLE CALCULATIONS NOT FOR DESIGN

Earth Mechanics · FUGRO



PRELIMINARY AXIAL PILE DESIGN PARAMETERS AND EXAMPLE RESULTS
Skyway Temporary Tower: BE and BW, Pier E2 to E3
0.41- and 0.61-Meter-Square Precast Concrete Piles
SFOBB East Span Seismic Safety Project

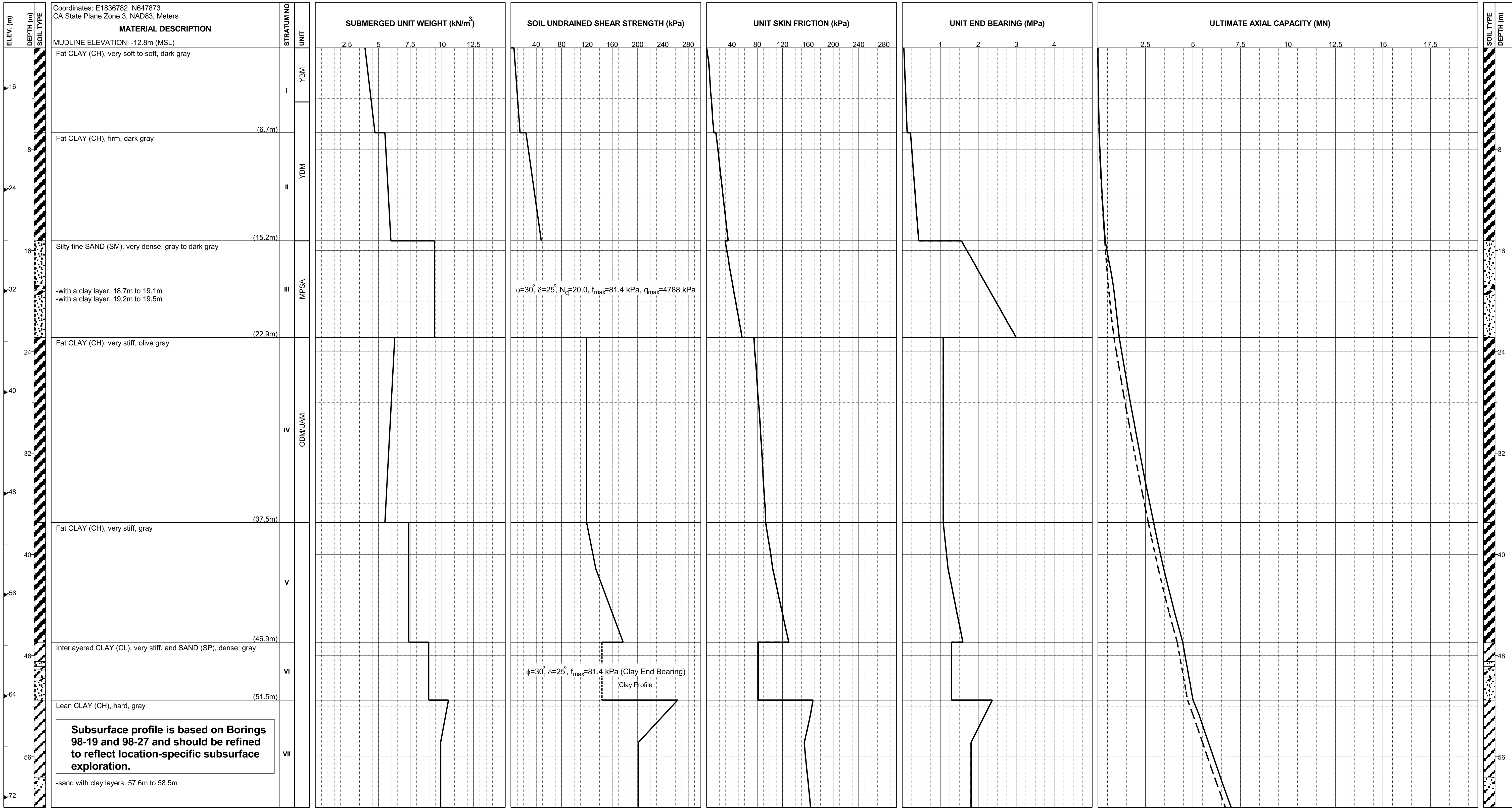
PLATE A1-B.2

SFOBB Task Order No. 5
Project No. 98-42-0054

EXAMPLE CALCULATIONS
NOT FOR DESIGN

Earth Mechanics

FUGRO



PRELIMINARY AXIAL PILE DESIGN PARAMETERS AND EXAMPLE RESULTS

Skyway Temporary Tower: BE and BW, Pier E2 to E3

0.36-Meter Steel H Pile

SFOBB East Span Seismic Safety Project

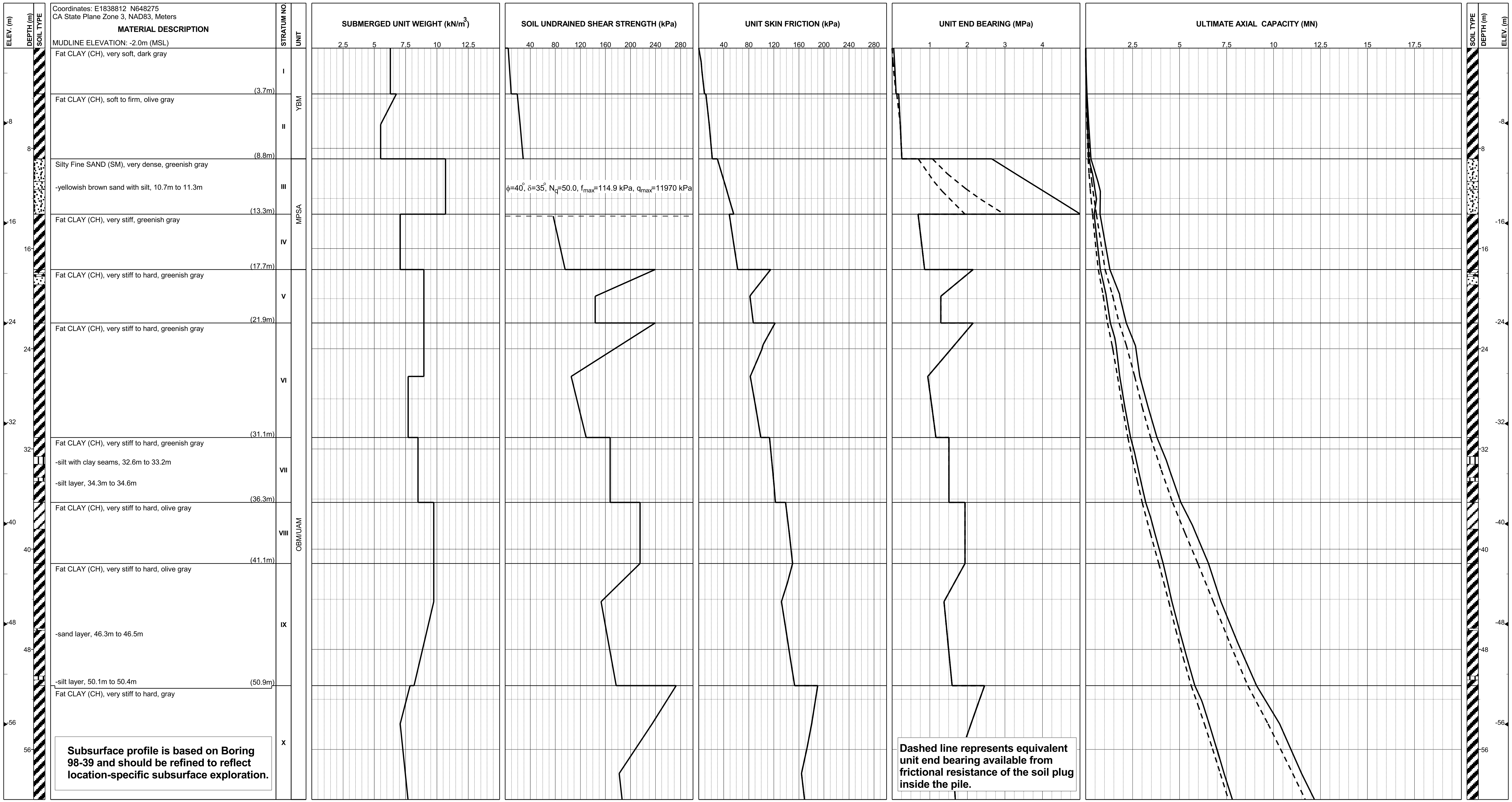
PLATE A1-B.3

SFOBB Task Order No. 5
Project No. 98-42-0054

EXAMPLE CALCULATIONS
NOT FOR DESIGN

Earth Mechanics

FUGRO



PRELIMINARY AXIAL PILE DESIGN PARAMETERS AND EXAMPLE RESULTS

Skyway Temporary Tower: CE and CW, Pier E16 to E17

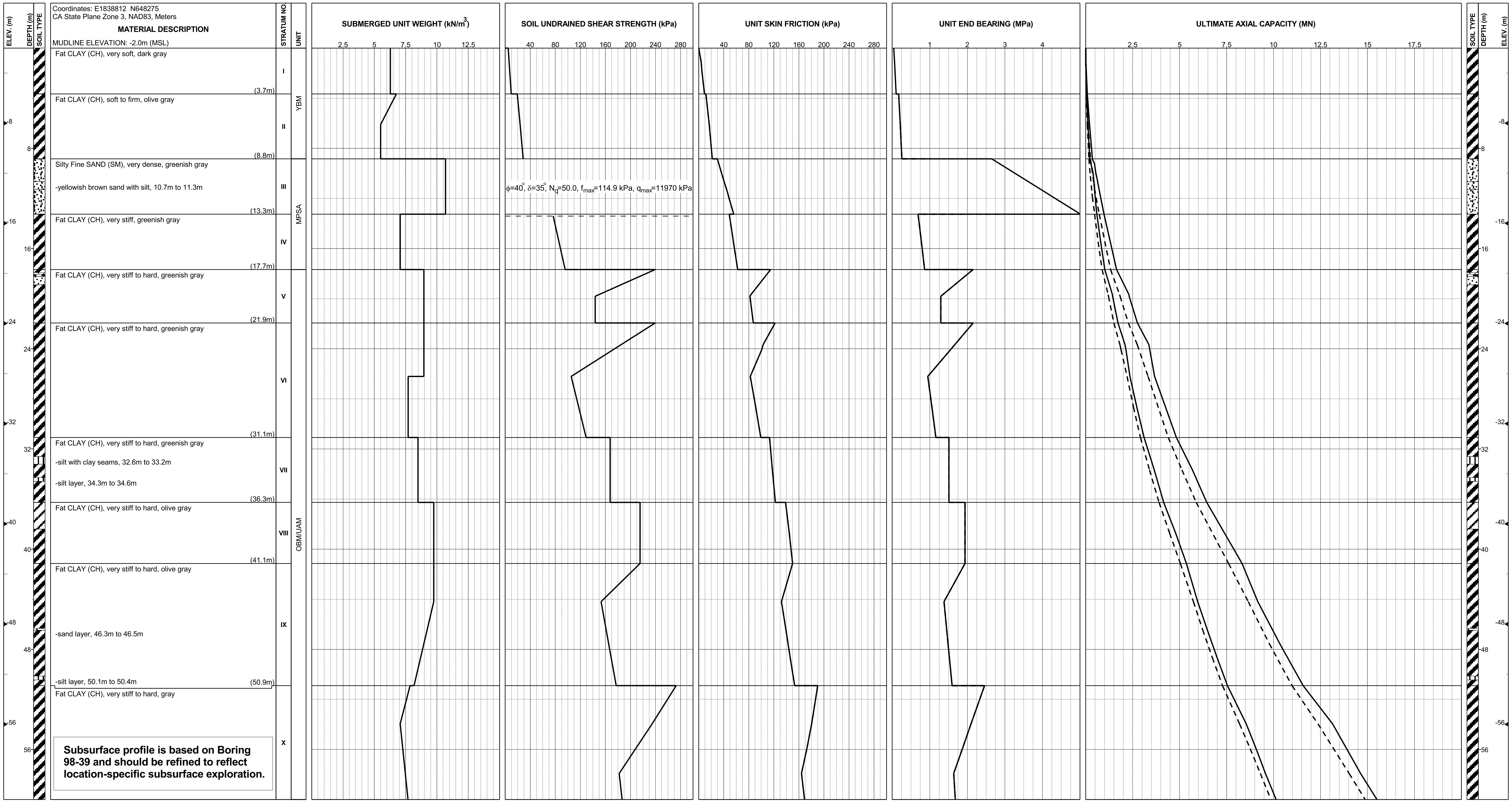
0.41- and 0.61-Meter-Diameter Steel Pipe Piles

SFOBB East Span Seismic Safety Project

SFOBB Task Order No. 5
Project No. 98-42-0054

EXAMPLE CALCULATIONS
NOT FOR DESIGN

Earth Mechanics
FUGRO



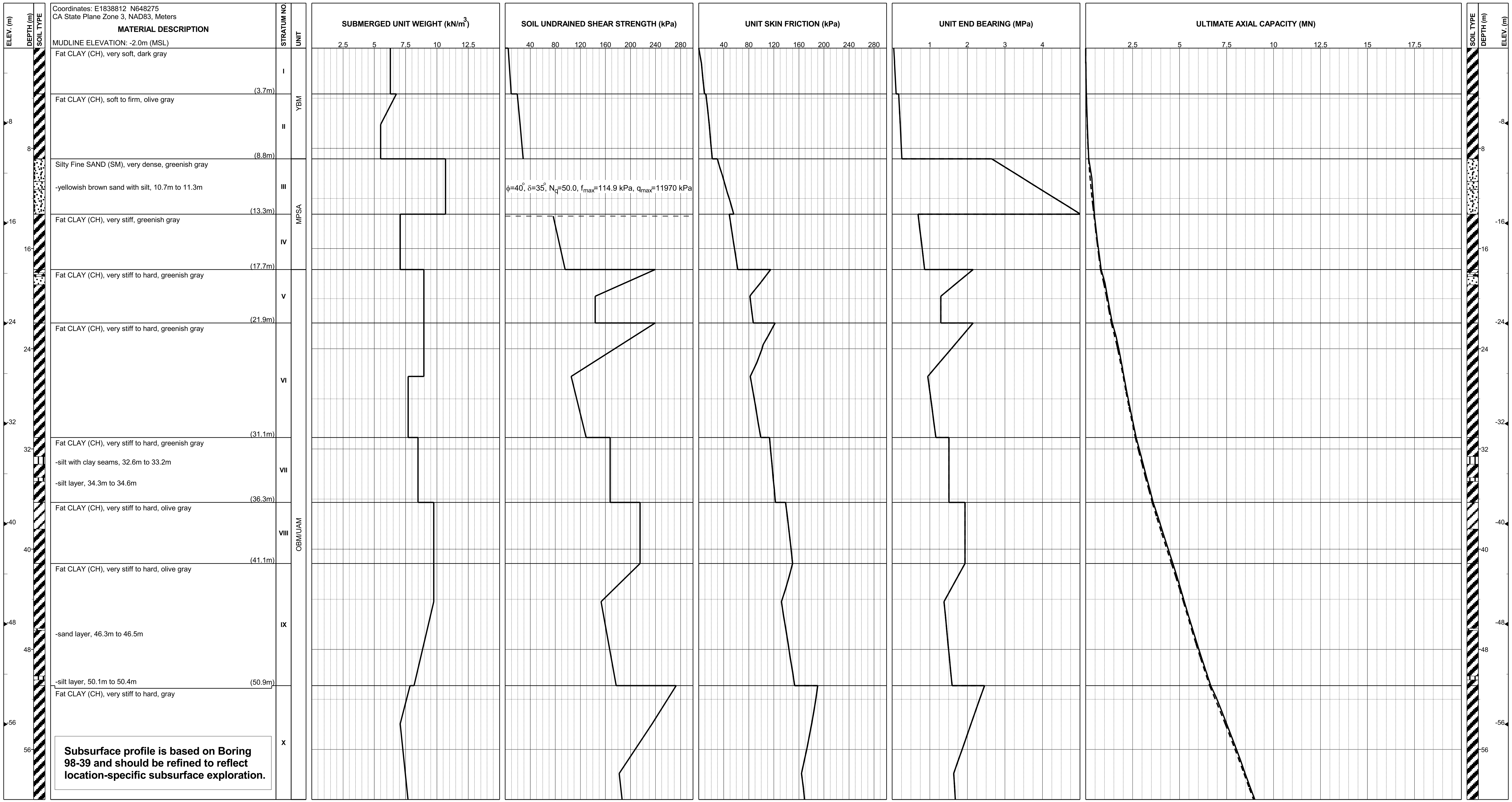
PRELIMINARY AXIAL PILE DESIGN PARAMETERS AND EXAMPLE RESULTS
Skyway Temporary Tower: CE and CW, Pier E16 to E17
0.41- and 0.61-Meter-Square Precast Concrete Piles
SFOBB East Span Seismic Safety Project

PLATE A1-C.2

SFOBB Task Order No. 5
Project No. 98-42-0054

EXAMPLE CALCULATIONS
NOT FOR DESIGN

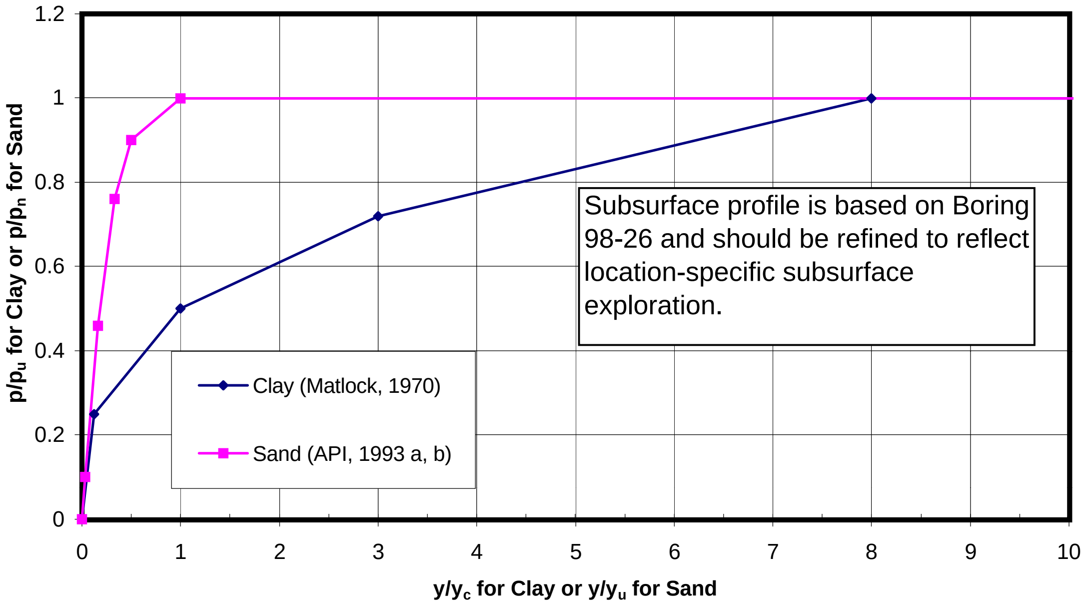
Earth Mechanics
FUGRO



PRELIMINARY AXIAL PILE DESIGN PARAMETERS AND EXAMPLE RESULTS
Skyway Temporary Tower: CE and CW, Pier E16 to E17
0.36-Meter Steel H Pile
SFOBB East Span Seismic Safety Project

PLATE A1-C.3

APPENDIX B
LATERAL LOAD-DEFLECTION ANALYSIS



Depth (m)	Soil Type	Clay		Sand	
		p_u, kN/m	y_c, m	p_n, kN/m	y_u, m
0.0	Clay	4.7	0.020		
0.4	Clay	7.6	0.020		
0.8	Clay	10.8	0.020		
1.2	Clay	14.2	0.020		
1.5	Clay	16.6	0.020		
1.5	Clay	16.5	0.020		
3.2	Clay	33.2	0.020		
8.2	Clay	69.8	0.020		
8.3	Clay	71.2	0.010		
9.7	Clay	130.0	0.010		
9.8	Clay	131.0	0.010		
16.4	Clay	157.0	0.010		
16.5	Clay	508.0	0.007		
19.8	Clay	368.0	0.007		
30.5	Clay	551.0	0.007		

Notes:

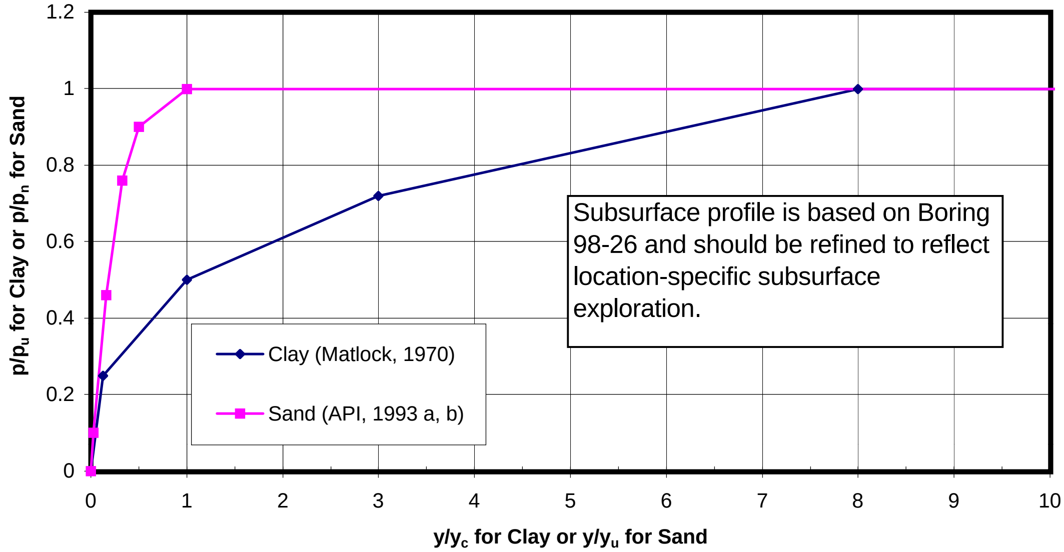
1. Mudline elevation = -12.7m (MSL)
2. The normalized curve for sand above is based on:

   p_n = A x p_u and y_u= (3 x p_n)/(k x Depth)

   where: A = (3-0.8 x (Depth/Pile Width))>=0.9 for static loading

   k = initial modulus of subgrade reaction
3. Interpolate p-y values for depths other than those provided.
4. For depths greater than 30.5 m below seafloor, use last values shown.

EXAMPLE LATERAL LOAD-DEFLECTION (p-y) CURVES
Skyway Temporary Tower: AE and AW, Pier E2 to E3
0.41-Meter-Diameter Pile
SFOBB East Span Seismic Safety Project



Depth (m)	Soil Type	Clay		Sand	
		p_u, kN/m	y_c, m	p_n, kN/m	y_u, m
0.0	Clay	7.01	0.031		
0.6	Clay	13.0	0.031		
1.2	Clay	19.3	0.031		
1.5	Clay	22.4	0.031		
1.5	Clay	22.2	0.031		
1.8	Clay	25.3	0.031		
4.8	Clay	68.7	0.031		
8.2	Clay	105	0.031		
8.3	Clay	107	0.015		
9.7	Clay	196	0.015		
9.8	Clay	187	0.015		
16.4	Clay	236	0.015		
16.5	Clay	640	0.011		
19.8	Clay	553	0.011		
30.5	Clay	827	0.011		

Notes:

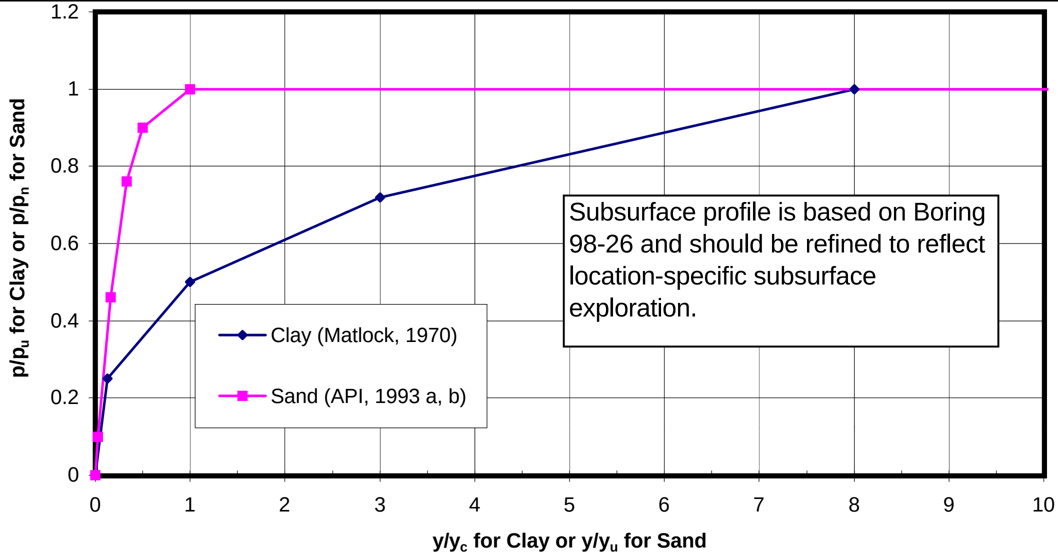
1. Mudline elevation = -12.7m (MSL)
2. The normalized curve for sand above is based on:

   p_n = A x p_u and y_u= (3 x p_n)/(k x Depth)

   where: A = (3-0.8 x (Depth/Pile Width))>=0.9 for static loading

   k = initial modulus of subgrade reaction
3. Interpolate p-y values for depths other than those provided.
4. For depths greater than 30.5 m below seafloor, use last values shown.

EXAMPLE LATERAL LOAD-DEFLECTION (p-y) CURVES
Skyway Temporary Tower: AE and AW, Pier E2 to E3
0.61-Meter-Diameter Pile
SFOBB East Span Seismic Safety Project



Depth (m)	Soil Type	Clay		Sand	
		p_u, kN/m	y_c, m	p_n, kN/m	y_u, m
0.0	Clay	4.1	0.018		
0.4	Clay	6.5	0.018		
0.7	Clay	9.0	0.018		
1.1	Clay	11.7	0.018		
1.5	Clay	15.1	0.018		
1.5	Clay	15.5	0.018		
2.8	Clay	27.5	0.018		
8.2	Clay	61.2	0.018		
8.3	Clay	62.4	0.009		
9.7	Clay	114.0	0.009		
9.8	Clay	115.0	0.009		
16.4	Clay	138.0	0.009		
16.5	Clay	459.0	0.006		
19.8	Clay	323.0	0.006		
30.5	Clay	483.0	0.006		

Notes:

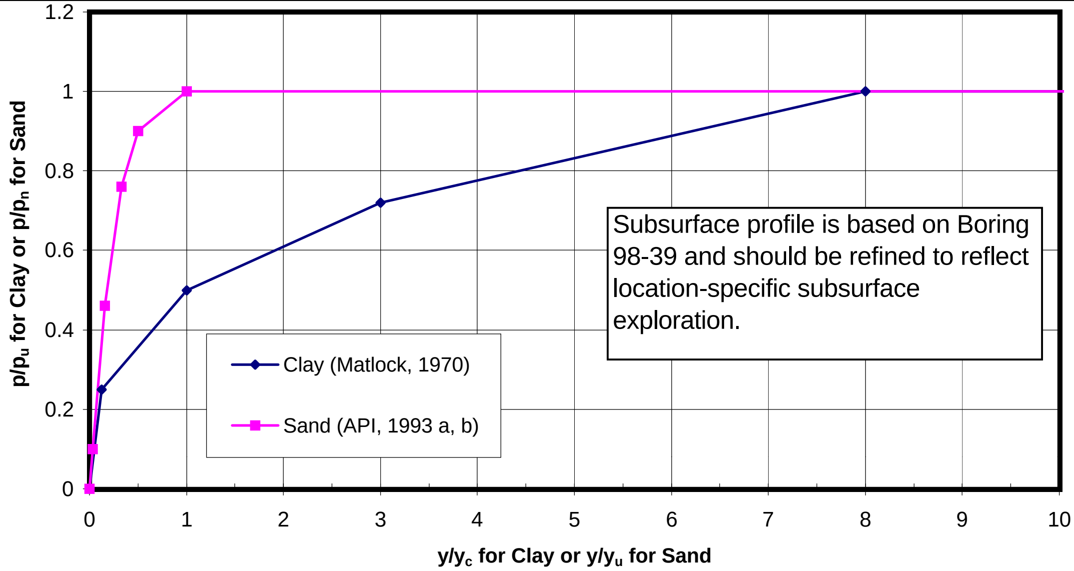
1. Mudline elevation = -12.7m (MSL)
2. The normalized curve for sand above is based on:

   p_n = A x p_u and y_u= (3 x p_n)/(k x Depth)

   where: A = (3-0.8 x (Depth/Pile Width))>=0.9 for static loading

   k = initial modulus of subgrade reaction
3. Interpolate p-y values for depths other than those provided.
4. For depths greater than 30.5 m below seafloor, use last values shown.

EXAMPLE LATERAL LOAD-DEFLECTION (p-y) CURVES
Skyway Temporary Tower: AE and AW, Pier E2 to E3
0.36-Meter Steel H Pile
SFOBB East Span Seismic Safety Project



Depth (m)	Soil Type	Clay		Sand	
		p_u, kN/m	y_c, m	p_n, kN/m	y_u, m
0.0	Clay	5.8	0.020		
0.4	Clay	8.9	0.020		
0.8	Clay	12.3	0.020		
1.2	Clay	16.0	0.020		
3.5	Clay	41.0	0.020		
3.6	Clay	41.8	0.020		
3.7	Clay	52.6	0.020		
6.1	Clay	86.3	0.020		
6.1	Clay	86.6	0.010		
8.8	Clay	105.0	0.010		
8.9	Sand			344.0	0.005
13.2	Sand			1300.0	0.012
13.3	Clay	280.0	0.007		
17.7	Clay	349.0	0.007		
17.7	Clay	871.0	0.007		
30.5	Clay	461.0	0.007		

Notes:

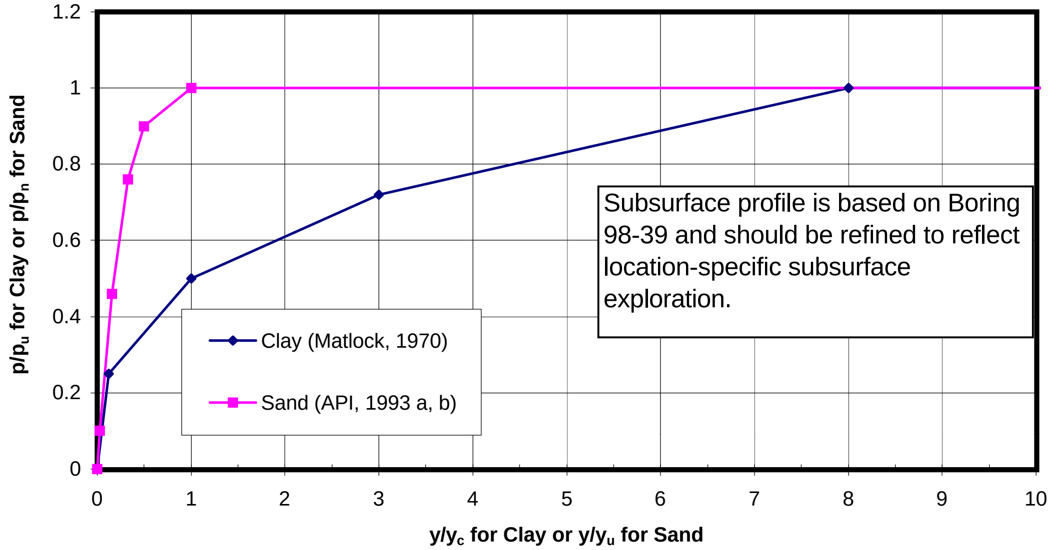
1. Mudline elevation = -2.0m (MSL)
2. The normalized curve for sand above is based on:

   p_n = A x p_u and y_u= (3 x p_n)/(k x Depth)

   where: A = (3-0.8 x (Depth/Pile Width))>=0.9 for static loading

   k = initial modulus of subgrade reaction
3. Interpolate p-y values for depths other than those provided.
4. For depths greater than 30.5 m below seafloor, use last values shown.

EXAMPLE LATERAL LOAD-DEFLECTION (p-y) CURVES
Skyway Temporary Tower: CE and CW, Pier E16 to E17
0.41-Meter-Diameter Pile
SFOBB East Span Seismic Safety Project



Depth (m)	Soil Type	Clay		Sand	
		p_u, kN/m	y_c, m	p_n, kN/m	y_u, m
0.0	Clay	8.8	0.031		
0.6	Clay	15.0	0.031		
1.2	Clay	21.8	0.031		
1.8	Clay	29.4	0.031		
3.6	Clay	55.6	0.031		
3.7	Clay	66.1	0.031		
5.5	Clay	103.0	0.031		
6.1	Clay	117.0	0.031		
6.1	Clay	111.0	0.015		
8.8	Clay	157.0	0.015		
8.9	Sand			417.0	0.006
13.2	Sand			1810.0	0.017
13.3	Clay	421.0	0.011		
17.7	Clay	525.0	0.011		
17.7	Clay	1310.0	0.011		
30.5	Clay	693.3	0.011		

Notes:

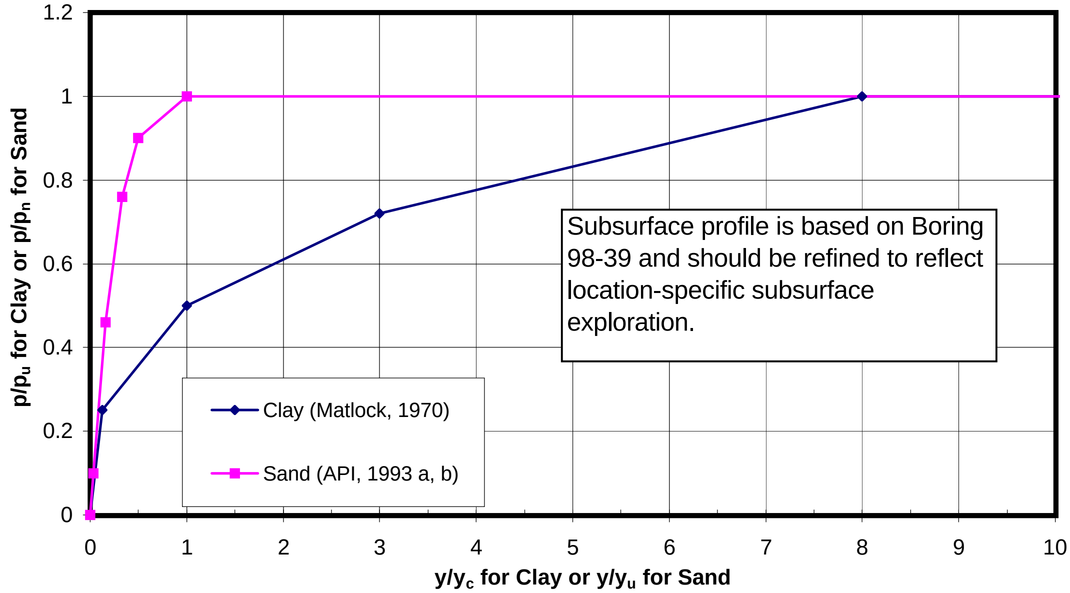
1. Mudline elevation = -2.0m (MSL)
2. The normalized curve for sand above is based on:

   p_n = A x p_u and y_u= (3 x p_n)/(k x Depth)

   where: A = (3-0.8 x (Depth/Pile Width))>=0.9 for static loading

   k = initial modulus of subgrade reaction
3. Interpolate p-y values for depths other than those provided.
4. For depths greater than 30.5 m below seafloor, use last values shown.

EXAMPLE LATERAL LOAD-DEFLECTION (p-y) CURVES
Skyway Temporary Tower: CE and CW, Pier E16 to E17
0.61-Meter-Diameter Pile
SFOBB East Span Seismic Safety Project



Depth (m)	Soil Type	Clay		Sand	
		p_u, kN/m	y_c, m	p_n, kN/m	y_u, m
0.0	Clay	5.1	0.018		
0.4	Clay	7.6	0.018		
0.7	Clay	10.3	0.018		
1.1	Clay	13.3	0.018		
3.1	Clay	33.4	0.018		
3.6	Clay	36.6	0.018		
3.7	Clay	48.2	0.018		
6.1	Clay	75.6	0.018		
6.1	Clay	75.9	0.009		
8.8	Clay	91.9	0.009		
8.9	Sand			325.0	0.004
13.2	Sand			1130.0	0.010
13.3	Clay	246.0	0.006		
17.7	Clay	306.0	0.006		
17.7	Clay	764.0	0.006		
30.5	Clay	405.0	0.006		

Notes:

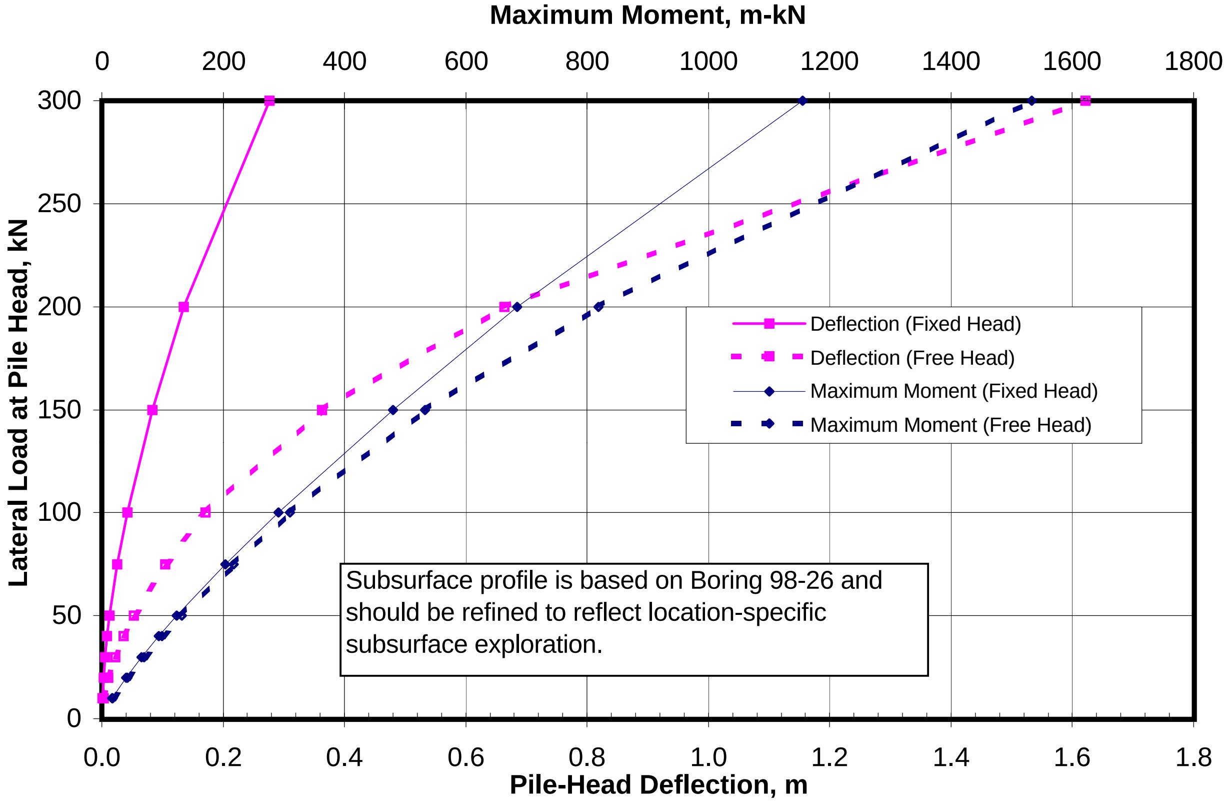
1. Mudline elevation = -2.0m (MSL)
2. The normalized curve for sand above is based on:

   p_n = A x p_u and y_u= (3 x p_n)/(k x Depth)

   where: A = (3-0.8 x (Depth/Pile Width))>=0.9 for static loading

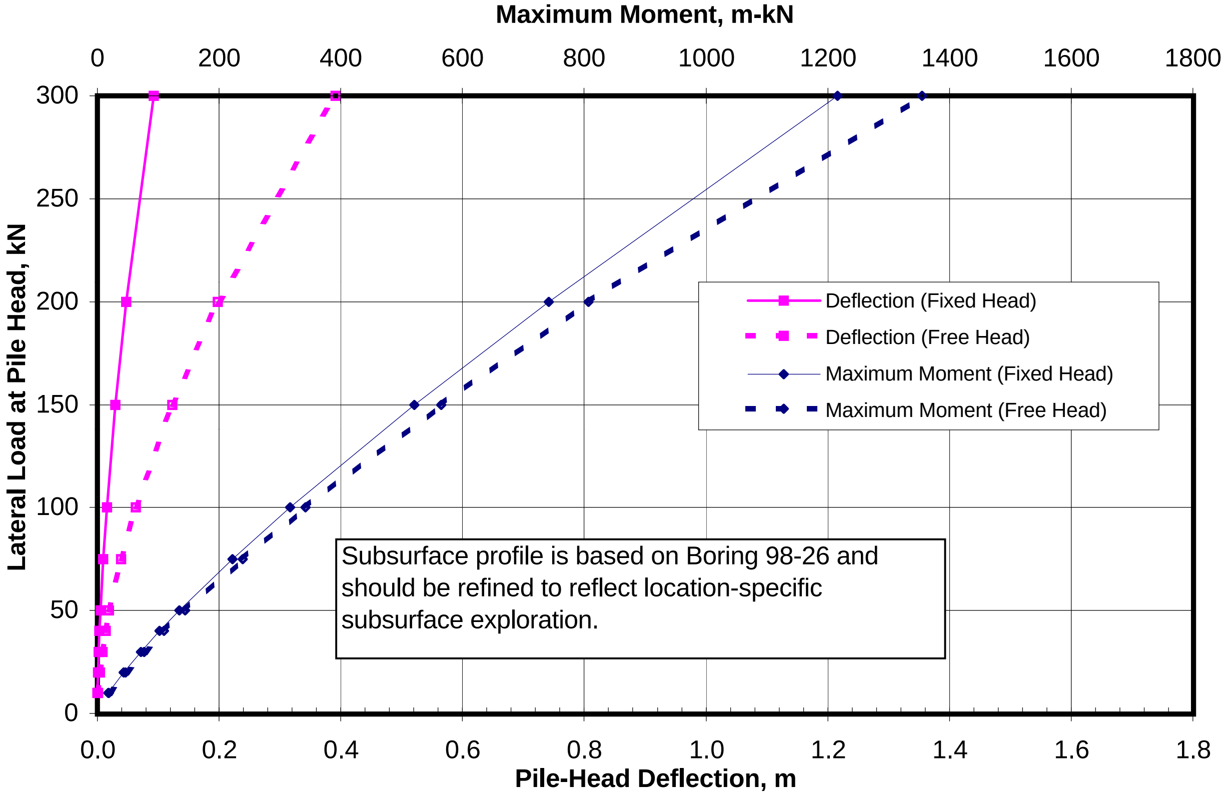
   k = initial modulus of subgrade reaction
3. Interpolate p-y values for depths other than those provided.
4. For depths greater than 30.5 m below seafloor, use last values shown.

EXAMPLE LATERAL LOAD-DEFLECTION (p-y) CURVES
Skyway Temporary Tower: CE and CW, Pier E16 to E17
0.36-Meter Steel H Pile
SFOBB East Span Seismic Safety Project



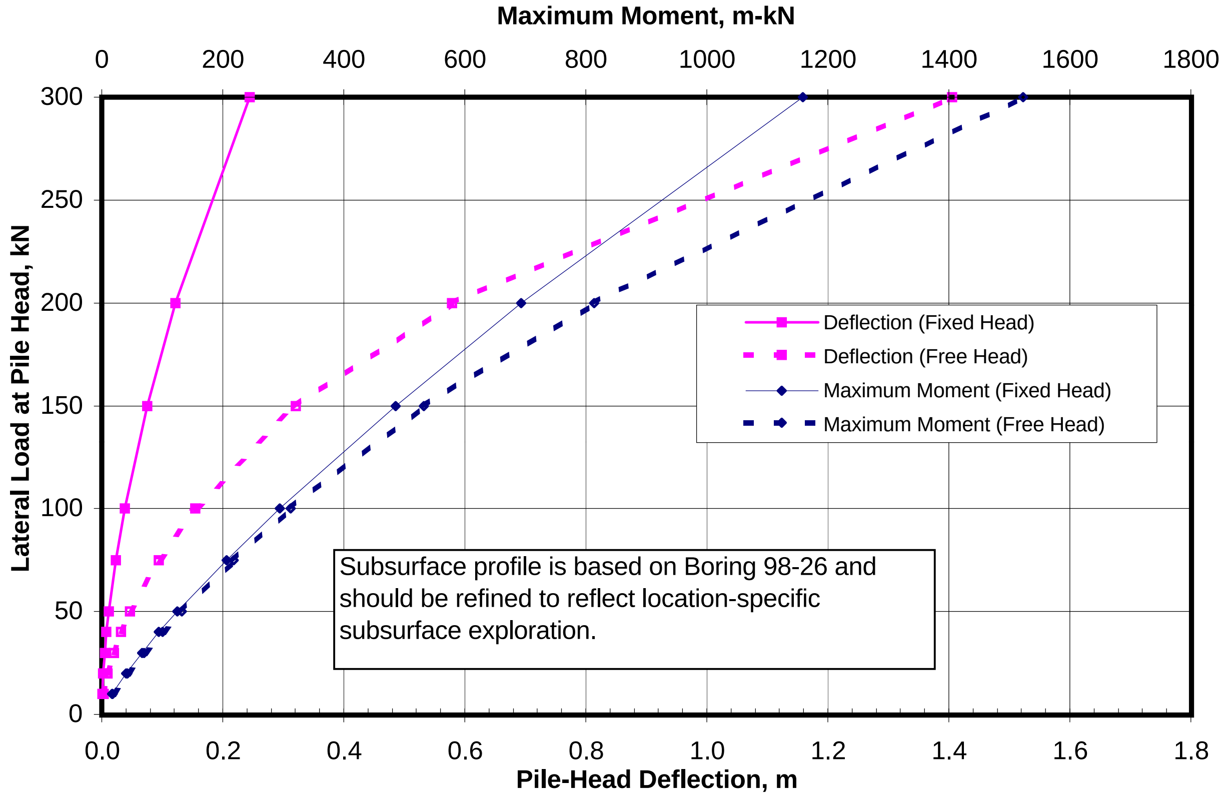
Lateral Load (kN)	Fixed Head		Free Head	
	Deflection (m)	Maximum Moment (m-kN)	Deflection (m)	Maximum Moment (m-kN)
10	0.001	17	0.003	18
20	0.003	40	0.011	42
30	0.005	66	0.022	70
40	0.009	94	0.036	100
50	0.013	123	0.052	132
75	0.026	204	0.105	217
100	0.042	291	0.171	310
150	0.083	480	0.363	533
200	0.135	685	0.664	819
300	0.276	1155	1.622	1533

EXAMPLE LATERAL PILE HEAD LOAD-DEFORMATION CURVES
Skyway Temporary Tower: AE and AW, Pier E2 to E3
0.41-Meter-Diameter Steel Pipe Pile
SFOBB East Span Seismic Safety Project



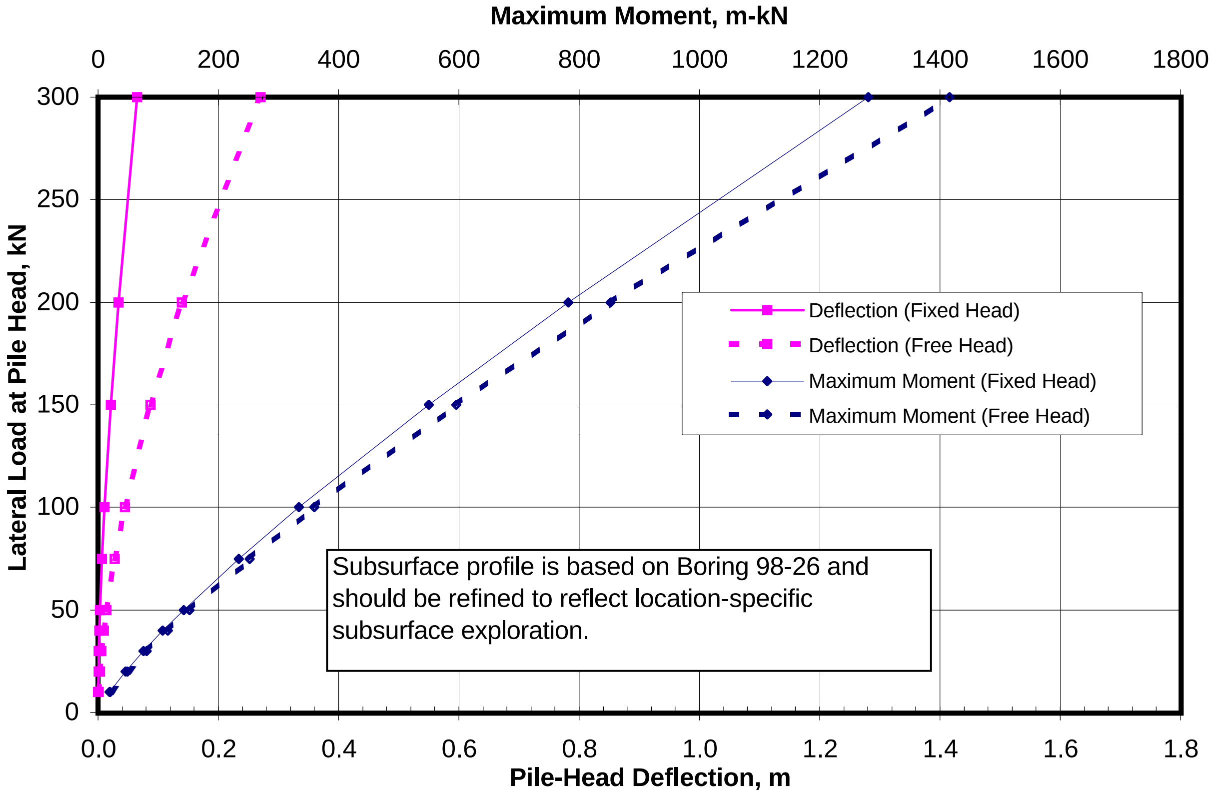
Lateral Load (kN)	Fixed Head		Free Head	
	Deflection (m)	Maximum Moment (m-kN)	Deflection (m)	Maximum Moment (m-kN)
10	0.000	18	0.001	19
20	0.001	43	0.004	46
30	0.002	72	0.008	76
40	0.003	102	0.013	109
50	0.005	135	0.019	144
75	0.009	222	0.039	239
100	0.015	317	0.063	342
150	0.030	521	0.123	565
200	0.048	742	0.198	807
300	0.092	1216	0.391	1355

EXAMPLE LATERAL PILE HEAD LOAD-DEFORMATION CURVES
Skyway Temporary Tower: AE and AW, Pier E2 to E3
0.61-Meter-Diameter Steel Pipe Pile
SFOBB East Span Seismic Safety Project



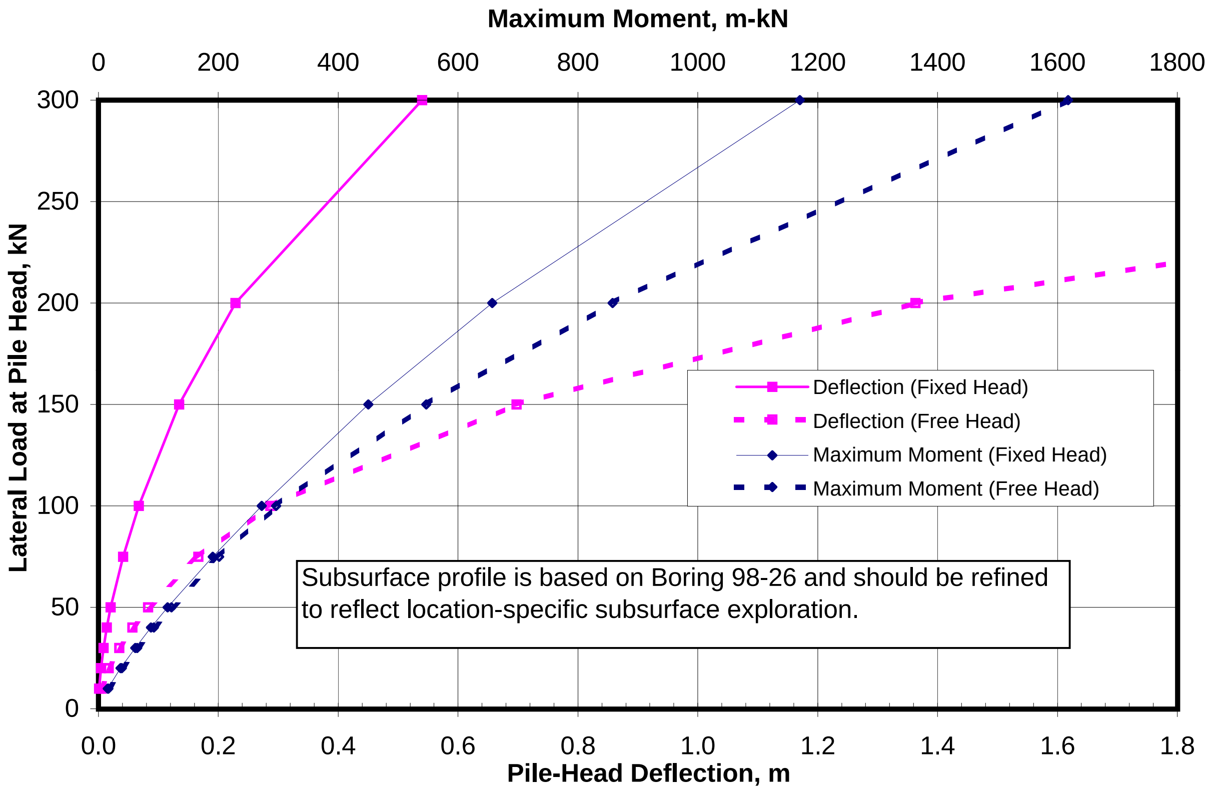
Lateral Load (kN)	Fixed Head		Free Head	
	Deflection (m)	Maximum Moment (m-kN)	Deflection (m)	Maximum Moment (m-kN)
10	0.001	17	0.003	18
20	0.002	40	0.010	42
30	0.005	66	0.020	70
40	0.008	95	0.032	100
50	0.012	125	0.047	133
75	0.023	206	0.094	219
100	0.038	294	0.154	312
150	0.075	486	0.321	532
200	0.122	693	0.579	814
300	0.245	1159	1.405	1523

EXAMPLE LATERAL PILE HEAD LOAD-DEFORMATION CURVES
Skyway Temporary Tower: AE and AW, Pier E2 to E3
0.41-Meter-Square Precast Concrete Pile
SFOBB East Span Seismic Safety Project



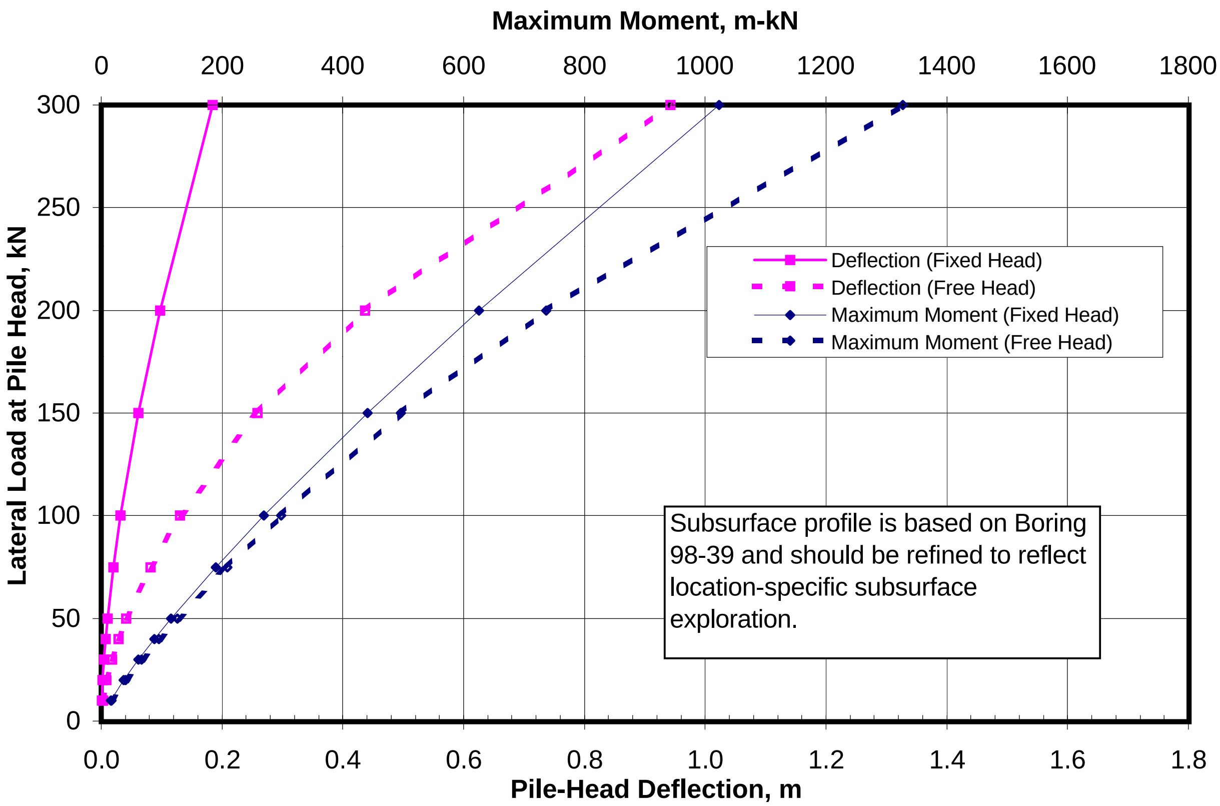
Lateral Load (kN)	Fixed Head		Free Head	
	Deflection (m)	Maximum Moment (m-kN)	Deflection (m)	Maximum Moment (m-kN)
10	0.000	19	0.001	20
20	0.001	46	0.003	49
30	0.001	76	0.006	81
40	0.002	108	0.009	116
50	0.003	142	0.014	153
75	0.007	234	0.027	252
100	0.011	334	0.044	360
150	0.021	550	0.087	596
200	0.034	781	0.140	853
300	0.065	1281	0.271	1416

EXAMPLE LATERAL PILE HEAD LOAD-DEFORMATION CURVES
Skyway Temporary Tower: AE and AW, Pier E2 to E3
0.61-Meter-Square Precast Concrete Pile
SFOBB East Span Seismic Safety Project



Lateral Load (kN)	Fixed Head		Free Head	
	Deflection (m)	Maximum Moment (m-kN)	Deflection (m)	Maximum Moment (m-kN)
10	0.001	16	0.005	16
20	0.004	37	0.017	39
30	0.009	61	0.035	65
40	0.014	87	0.057	92
50	0.020	115	0.083	122
75	0.041	191	0.167	201
100	0.067	272	0.287	296
150	0.135	450	0.697	547
200	0.229	657	1.363	858
300	0.540	1170	3.564	1618

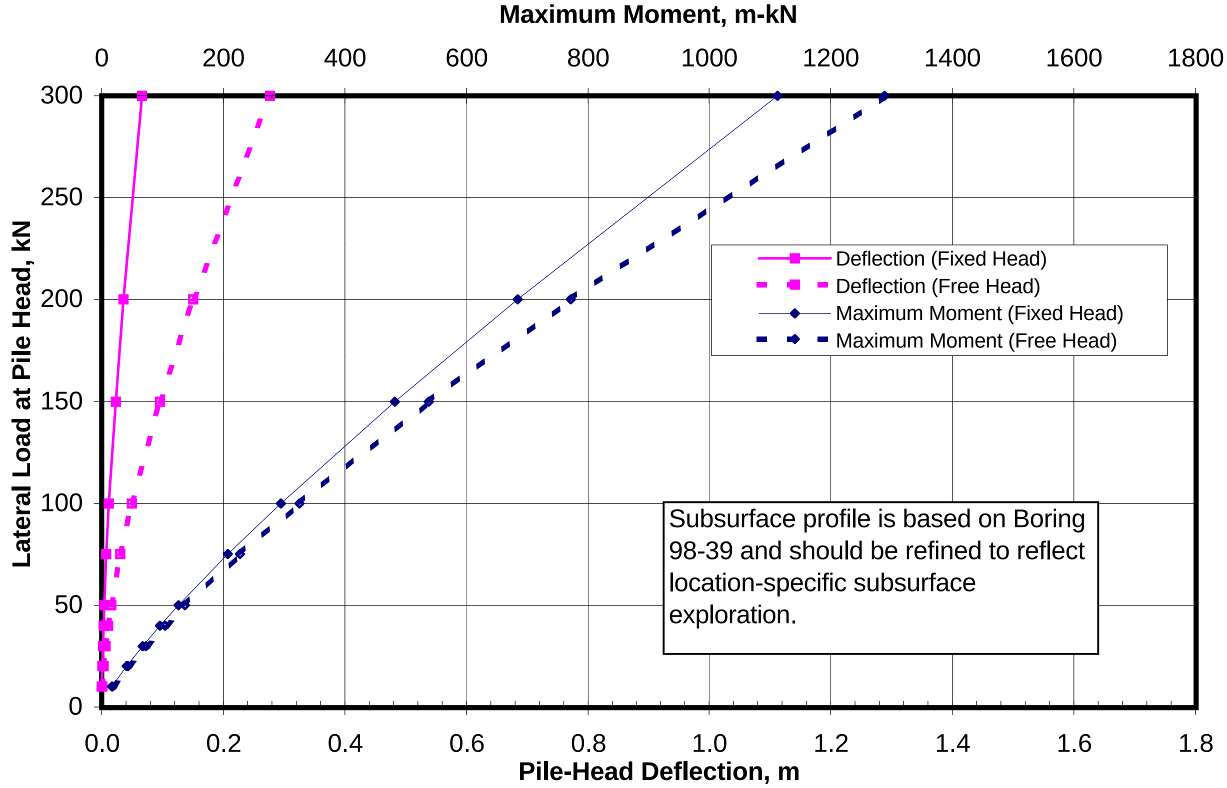
EXAMPLE LATERAL PILE HEAD LOAD-DEFORMATION CURVES
Skyway Temporary Tower: AE and AW, Pier E2 to E3
0.36-Meter Steel H Pile
SFOBB East Span Seismic Safety Project



Lateral Load (kN)	Fixed Head		Free Head	
	Deflection (m)	Maximum Moment (m-kN)	Deflection (m)	Maximum Moment (m-kN)
10	0.001	16	0.003	17
20	0.002	37	0.009	40
30	0.004	62	0.018	67
40	0.007	88	0.029	95
50	0.010	115	0.042	126
75	0.020	190	0.081	209
100	0.032	269	0.131	298
150	0.061	441	0.259	496
200	0.098	626	0.437	736
300	0.184	1023	0.942	1327

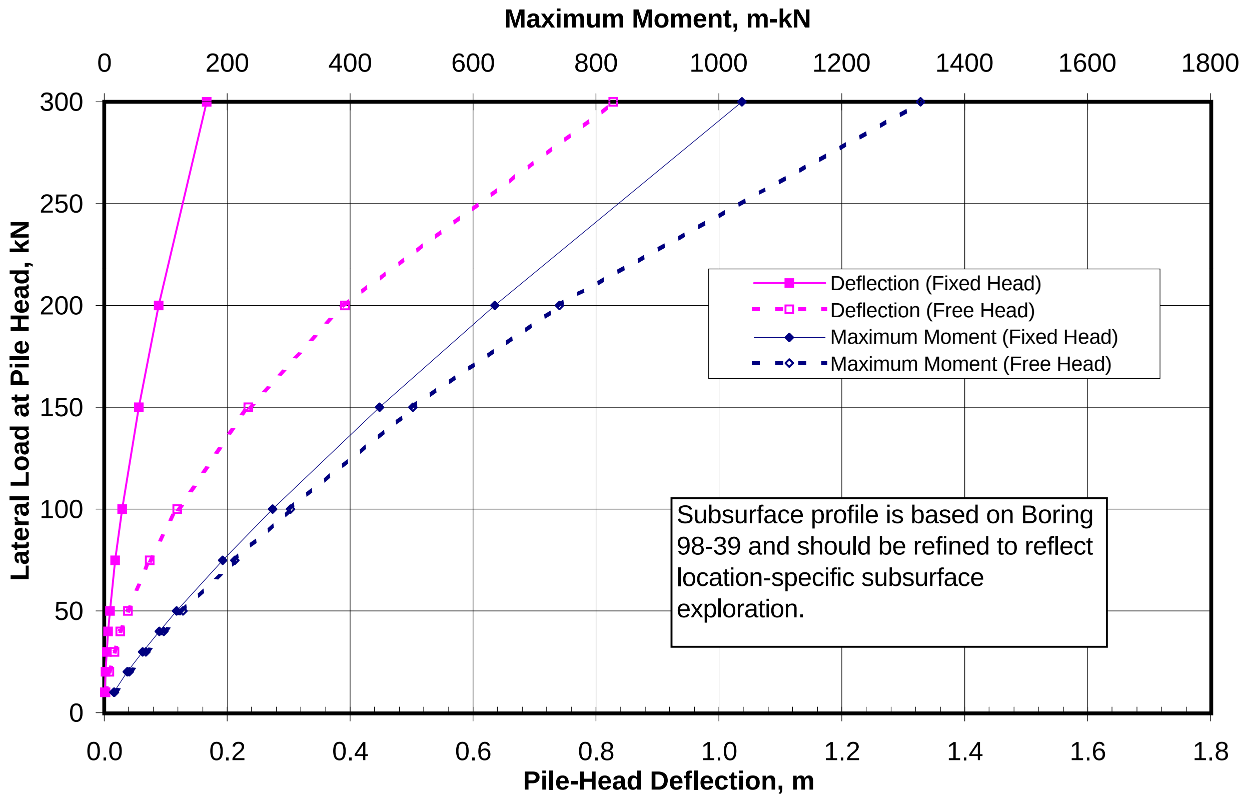
EXAMPLE LATERAL PILE HEAD LOAD-DEFORMATION CURVES
Skyway Temporary Tower: CE and CW, Pier E16 to E17
0.41-Meter-Diameter Steel Pipe Pile
SFOBB East Span Seismic Safety Project

EXAMPLE CALCULATIONS
NOT FOR DESIGN



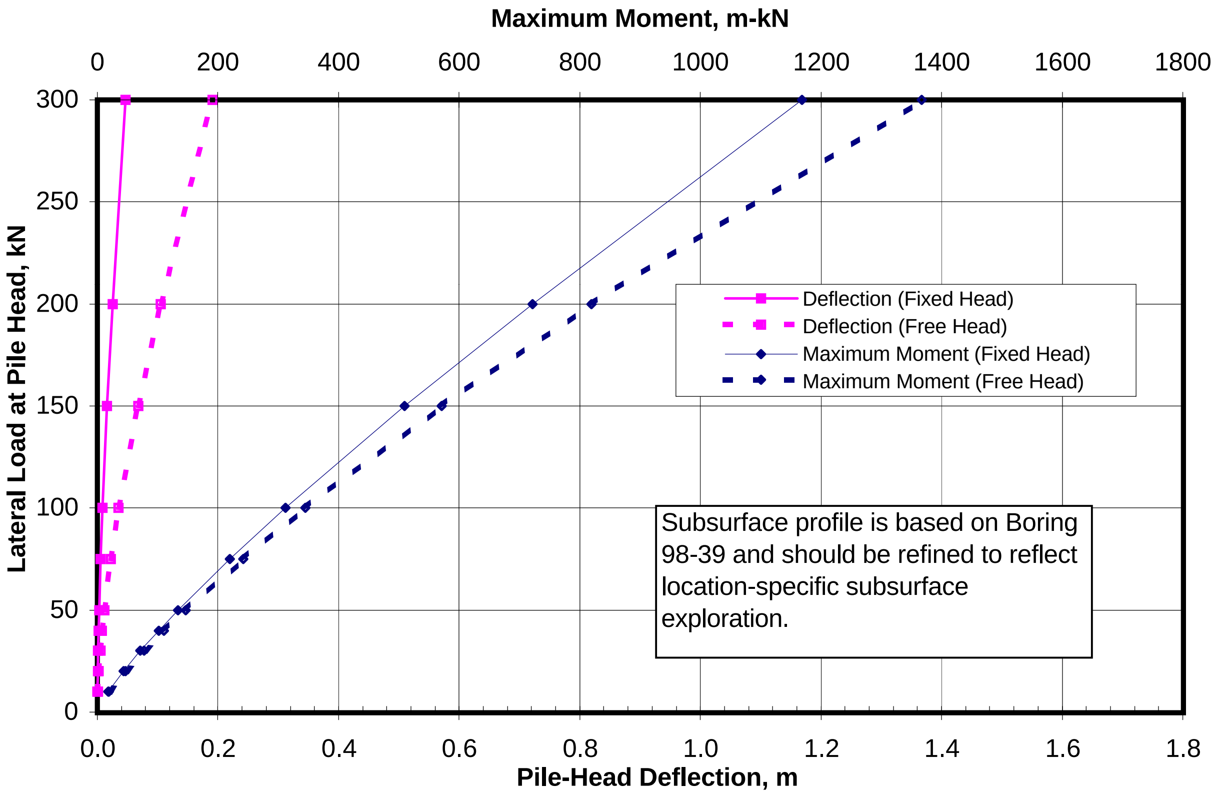
Lateral Load (kN)	Fixed Head		Free Head	
	Deflection (m)	Maximum Moment (m-kN)	Deflection (m)	Maximum Moment (m-kN)
10	0.000	17	0.001	18
20	0.001	41	0.003	44
30	0.002	67	0.007	73
40	0.003	96	0.011	104
50	0.004	126	0.016	137
75	0.007	208	0.031	228
100	0.012	295	0.049	325
150	0.023	483	0.096	538
200	0.036	684	0.150	772
300	0.067	1112	0.277	1288

EXAMPLE LATERAL PILE HEAD LOAD-DEFORMATION CURVES
Skyway Temporary Tower: CE and CW, Pier E16 to E17
0.61-Meter-Diameter Steel Pipe Pile
SFOBB East Span Seismic Safety Project



Lateral Load (kN)	Fixed Head		Free Head	
	Deflection (m)	Maximum Moment (m-kN)	Deflection (m)	Maximum Moment (m-kN)
10	0.001	16	0.003	17
20	0.002	38	0.008	41
30	0.004	63	0.016	68
40	0.006	89	0.026	97
50	0.009	117	0.038	128
75	0.018	193	0.074	212
100	0.029	274	0.119	303
150	0.056	448	0.235	502
200	0.089	636	0.391	741
300	0.167	1037	0.828	1328

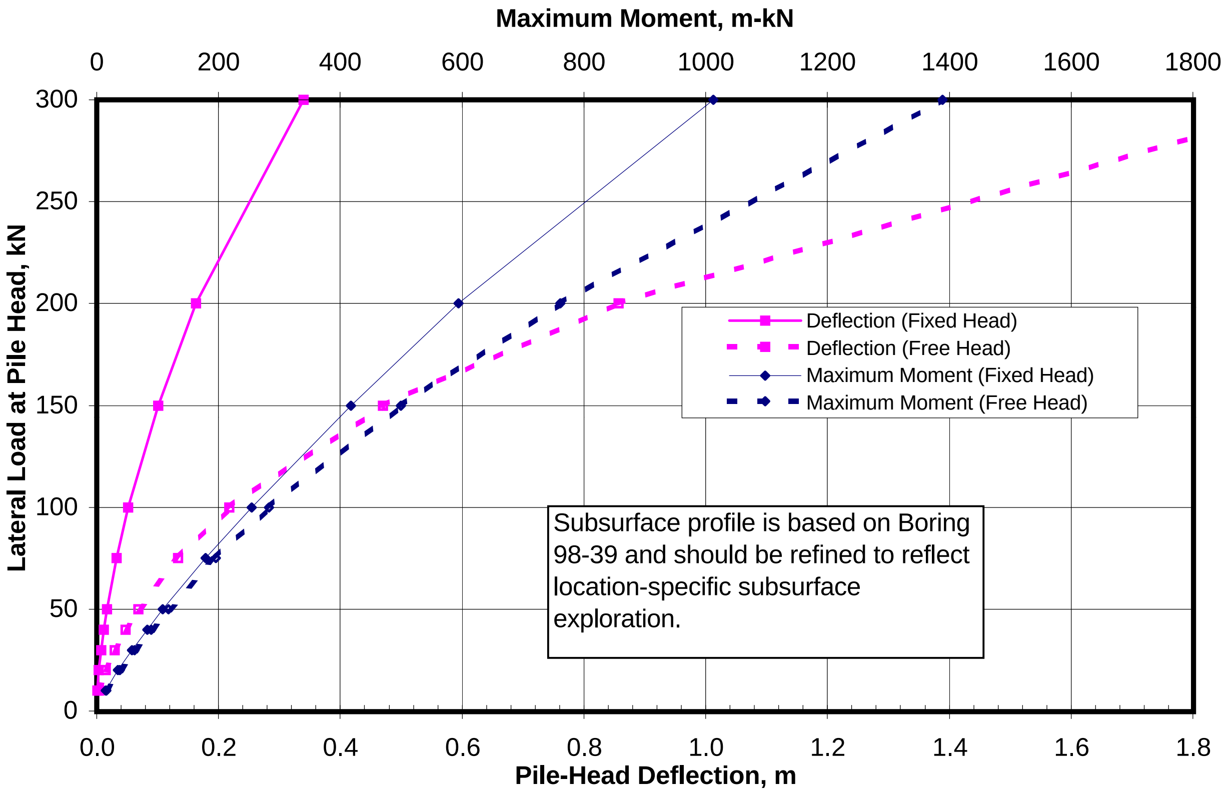
EXAMPLE LATERAL PILE HEAD LOAD-DEFORMATION CURVES
Skyway Temporary Tower: CE and CW, Pier E16 to E17
0.41-Meter-Square Precast Concrete Pile
SFOBB East Span Seismic Safety Project



Lateral Load (kN)	Fixed Head Deflection (m)	Fixed Head Maximum Moment (m-kN)	Free Head Deflection (m)	Free Head Maximum Moment (m-kN)
10	0.000	18	0.001	19
20	0.001	43	0.002	46
30	0.001	71	0.005	77
40	0.002	102	0.008	111
50	0.003	134	0.011	146
75	0.005	220	0.022	242
100	0.008	312	0.035	345
150	0.016	510	0.067	572
200	0.025	721	0.105	820
300	0.046	1169	0.191	1367

EXAMPLE LATERAL PILE HEAD LOAD-DEFORMATION CURVES
Skyway Temporary Tower: CE and CW, Pier E16 to E17
0.61-Meter-Square Precast Concrete Pile
SFOBB East Span Seismic Safety Project

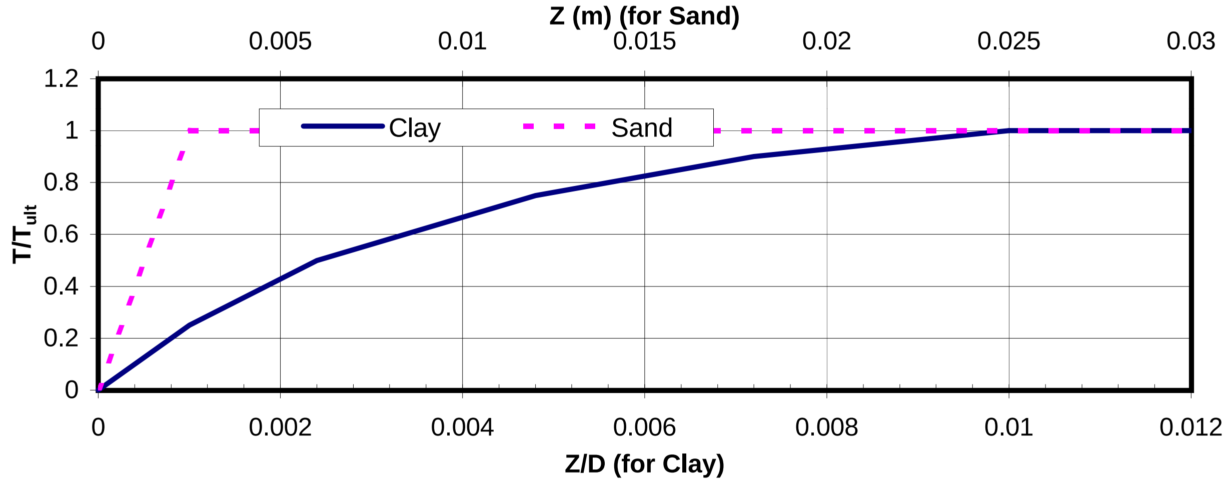
EXAMPLE CALCULATIONS
NOT FOR DESIGN



Lateral Load (kN)	Fixed Head		Free Head	
	Deflection (m)	Maximum Moment (m-kN)	Deflection (m)	Maximum Moment (m-kN)
10	0.001	15	0.004	16
20	0.004	35	0.015	37
30	0.007	58	0.029	62
40	0.011	83	0.047	90
50	0.017	109	0.068	118
75	0.032	179	0.133	196
100	0.052	254	0.218	283
150	0.101	417	0.470	499
200	0.163	594	0.856	761
300	0.340	1013	2.012	1389

EXAMPLE LATERAL PILE HEAD LOAD-DEFORMATION CURVES
Skyway Temporary Tower: CE and CW, Pier E16 to E17
0.36-Meter Steel H Pile
SFOBB East Span Seismic Safety Project

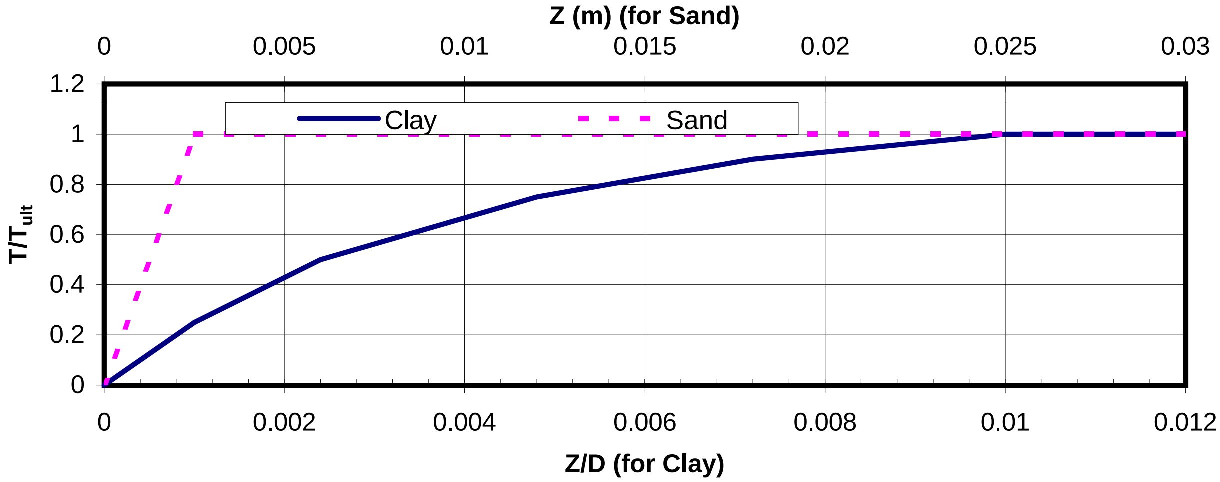
APPENDIX C
AXIAL LOAD-DEFLECTION ANALYSIS



Subsurface profile is based on Boring 98-26 and should be refined to reflect location-specific subsurface exploration.

Depth (m)	Soil Type	T_{ult} (kPa)
0.0	Clay	0.0
1.5	Clay	4.3
8.2	Clay	16.3
9.8	Clay	24.4
16.5	Clay	35.9
16.5	Clay	72.3
19.8	Clay	58.9
19.8	Clay	61.8
21.6	Clay	69.4
31.1	Clay	96.7
31.1	Clay	90.5
38.4	Clay	102.0
38.4	Clay	115.4
38.7	Clay	115.9

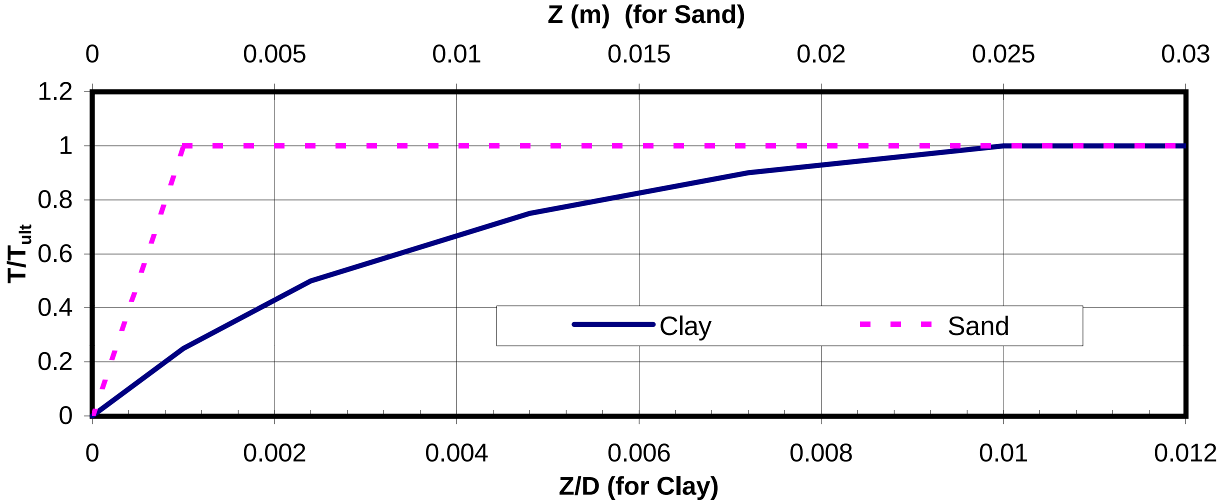
EXAMPLE AXIAL PILE LOAD TRANSFER-DISPLACEMENT CURVES
Skyway Temporary Tower: AE and AW, Pier E2 to E3
0.41-Meter-Diameter Steel Pipe Pile (Pile Tip Depth = 38.7 meters)
SFOBB East Span Seismic Safety Project



Subsurface profile is based on Boring 98-26 and should be refined to reflect location-specific subsurface exploration.

Depth (m)	Soil Type	T_{ult} (kPa)
0.0	Clay	0.0
1.5	Clay	4.3
8.2	Clay	16.3
9.8	Clay	24.4
16.5	Clay	35.9
16.5	Clay	72.3
19.8	Clay	58.9
19.8	Clay	61.8
21.6	Clay	69.4
30.8	Clay	95.8

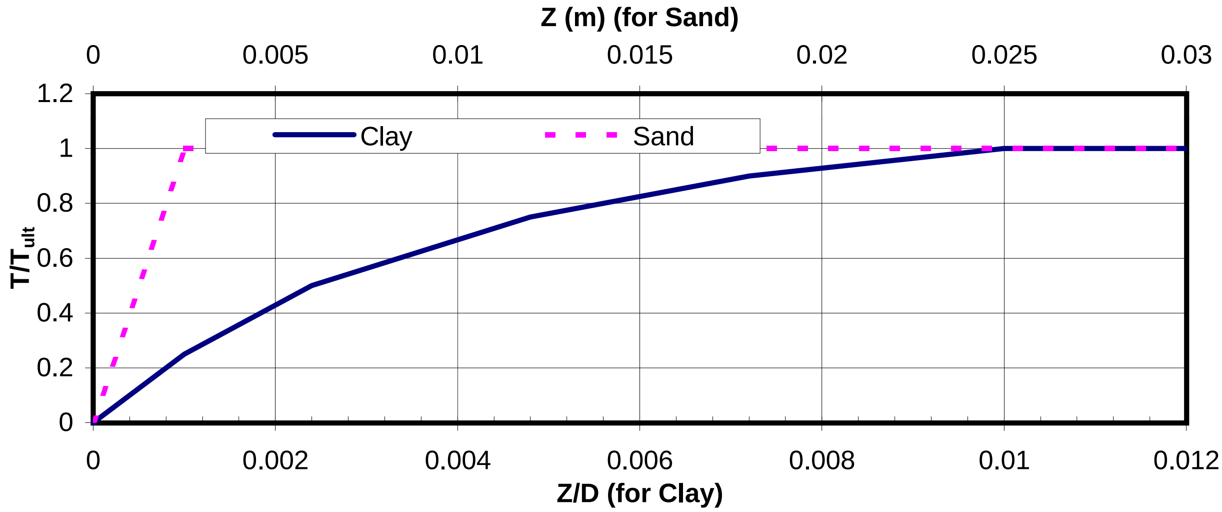
EXAMPLE AXIAL PILE LOAD TRANSFER-DISPLACEMENT CURVES
Skyway Temporary Tower: AE and AW, Pier E2 to E3
0.61-Meter-Diameter Steel Pipe Pile (Pile Tip Depth = 30.8 meters)
SFOBB East Span Seismic Safety Project



Subsurface profile is based on Boring 98-26 and should be refined to reflect location-specific subsurface exploration.

Depth (m)	Soil Type	T_{ult} (kPa)
0.0	Clay	0.0
1.5	Clay	4.3
8.2	Clay	16.3
9.8	Clay	24.4
16.5	Clay	35.9
16.5	Clay	72.3
19.8	Clay	58.9
19.8	Clay	61.8
21.6	Clay	69.4
31.1	Clay	96.7
31.1	Clay	90.5
33.5	Clay	94.3

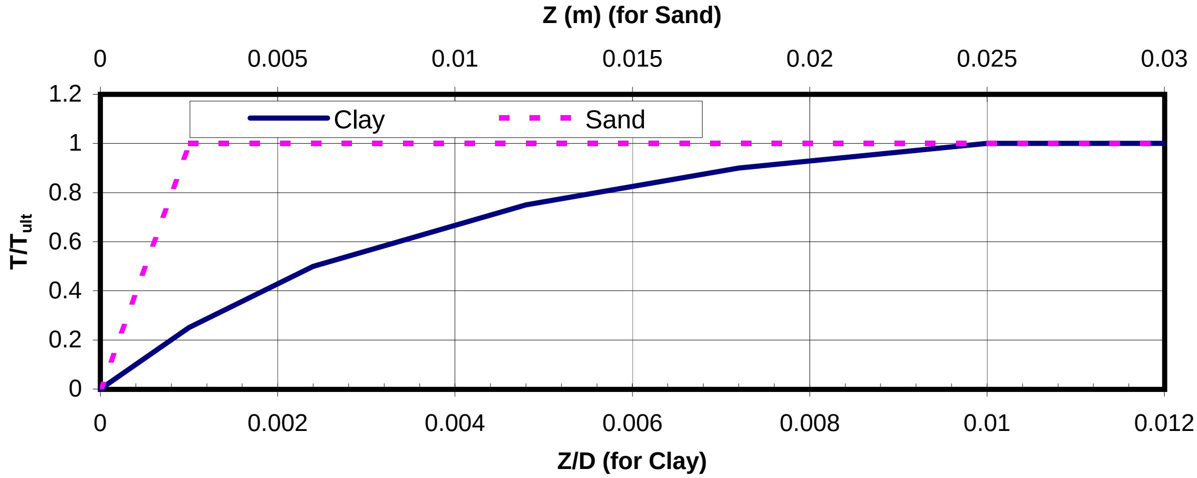
EXAMPLE AXIAL PILE LOAD TRANSFER-DISPLACEMENT CURVES
Skyway Temporary Tower: AE and AW, Pier E2 to E3
0.41-Meter-Square Precast Concrete Pile (Pile Tip Depth = 33.5 meters)
SFOBB East Span Seismic Safety Project



Subsurface profile is based on Boring 98-26 and should be refined to reflect location-specific subsurface exploration.

Depth (m)	Soil Type	T_{ult} (kPa)
0.0	Clay	0.0
1.5	Clay	4.3
8.2	Clay	16.3
9.8	Clay	24.4
16.5	Clay	35.9
16.5	Clay	72.3
19.8	Clay	58.9
19.8	Clay	61.8
21.6	Clay	69.4
27.4	Clay	85.7

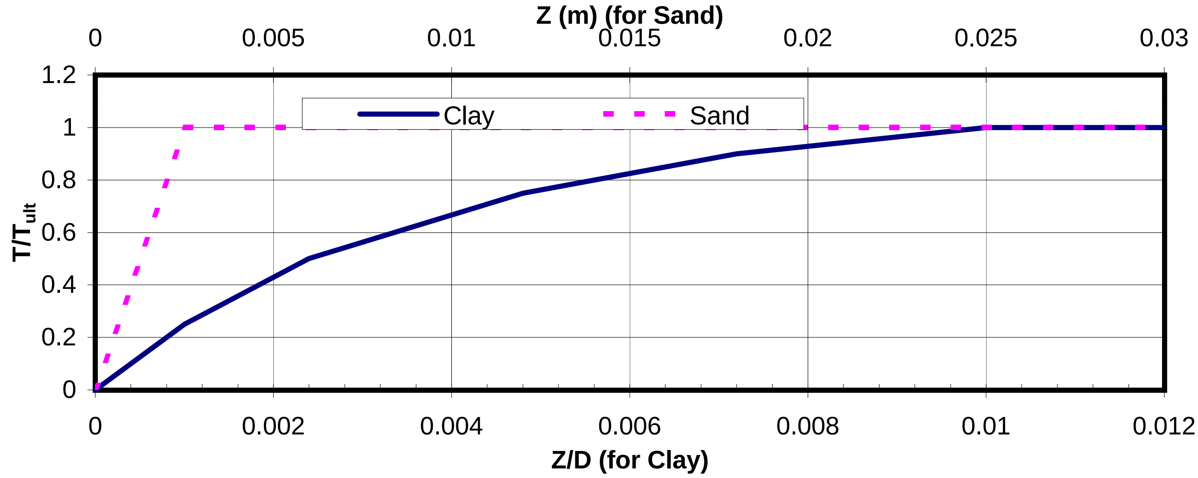
EXAMPLE AXIAL PILE LOAD TRANSFER-DISPLACEMENT CURVES
Skyway Temporary Tower: AE and AW, Pier E2 to E3
0.61-Meter-Square Precast Concrete Pile (Pile Tip Depth = 27.4 meters)
SFOBB East Span Seismic Safety Project



Subsurface profile is based on Boring 98-26 and should be refined to reflect location-specific subsurface exploration.

Depth (m)	Soil Type	T_{ult} (kPa)
0.0	Clay	0.0
1.5	Clay	4.3
8.2	Clay	16.3
9.8	Clay	24.4
16.5	Clay	35.9
16.5	Clay	72.3
19.8	Clay	58.9
19.8	Clay	61.8
21.6	Clay	69.4
31.1	Clay	96.7
31.1	Clay	90.5
35.4	Clay	97.2

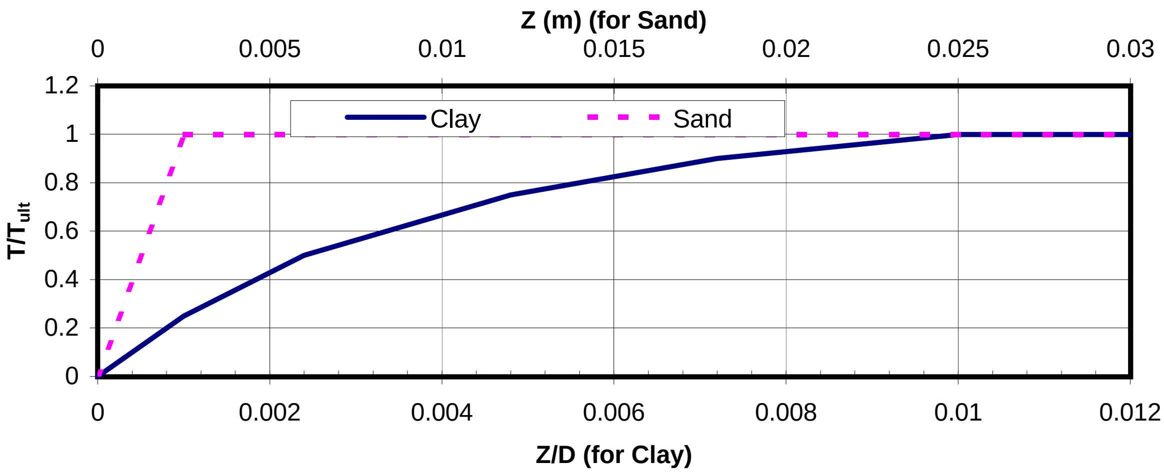
EXAMPLE AXIAL PILE LOAD TRANSFER-DISPLACEMENT CURVES
Skyway Temporary Tower: AE and AW, Pier E2 to E3
0.36-Meter Steel H Pile (Pile Tip Depth = 35.4 meters)
SFOBB East Span Seismic Safety Project



Subsurface profile is based on Boring 98-39 and should be refined to reflect location-specific subsurface exploration.

Depth (m)	Soil Type	T_{ult} (kPa)
0.0	Clay	0.0
3.7	Clay	9.1
3.7	Clay	11.5
6.1	Clay	16.8
8.8	Clay	21.5
8.8	Sand	29.7
13.3	Sand	56.0
13.3	Clay	48.8
17.7	Clay	62.2
17.7	Clay	114.9
19.8	Clay	81.9
21.9	Clay	87.1
21.9	Clay	122.1
26.2	Clay	82.4
31.1	Clay	99.1
31.1	Clay	113.0
34.1	Clay	118.7

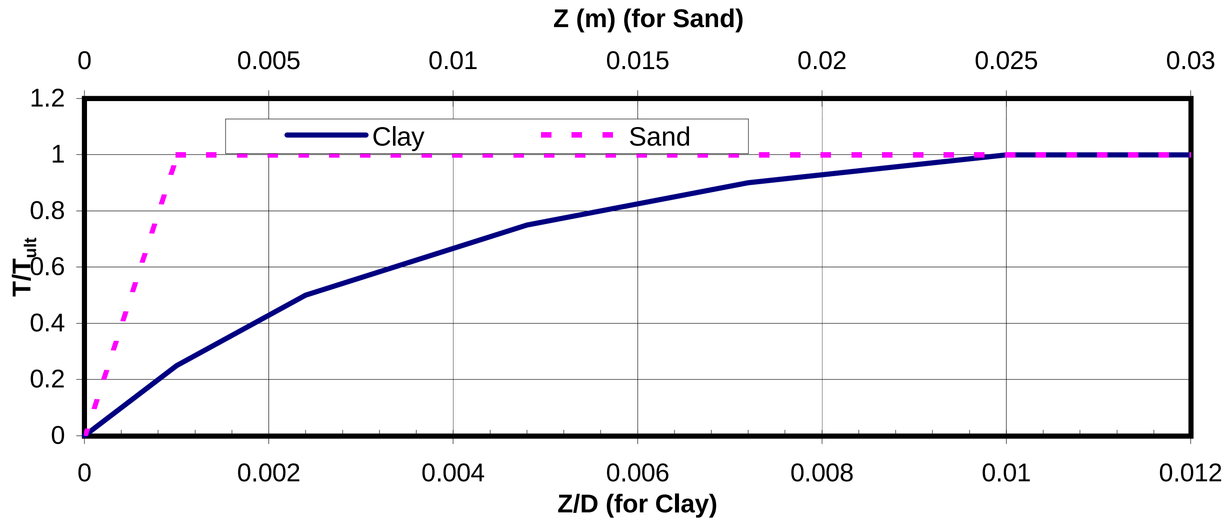
EXAMPLE AXIAL PILE LOAD TRANSFER-DISPLACEMENT CURVES
Skyway Temporary Tower: CE and CW, Pier E16 to E17
0.41-Meter-Diameter Steel Pipe Pile (Pile Tip Depth = 34.1 meters)
SFOBB East Span Seismic Safety Project



Subsurface profile is based on Boring 98-39 and should be refined to reflect location-specific subsurface exploration.

Depth (m)	Soil Type	T_{ult} (kPa)
0.0	Clay	0.0
3.7	Clay	9.1
3.7	Clay	11.5
6.1	Clay	16.8
8.8	Clay	21.5
8.8	Sand	29.7
13.3	Sand	56.0
13.3	Clay	48.8
17.7	Clay	62.2
17.7	Clay	114.9
19.8	Clay	81.9
21.9	Clay	87.1
21.9	Clay	122.1
26.2	Clay	82.4
26.8	Clay	84.7

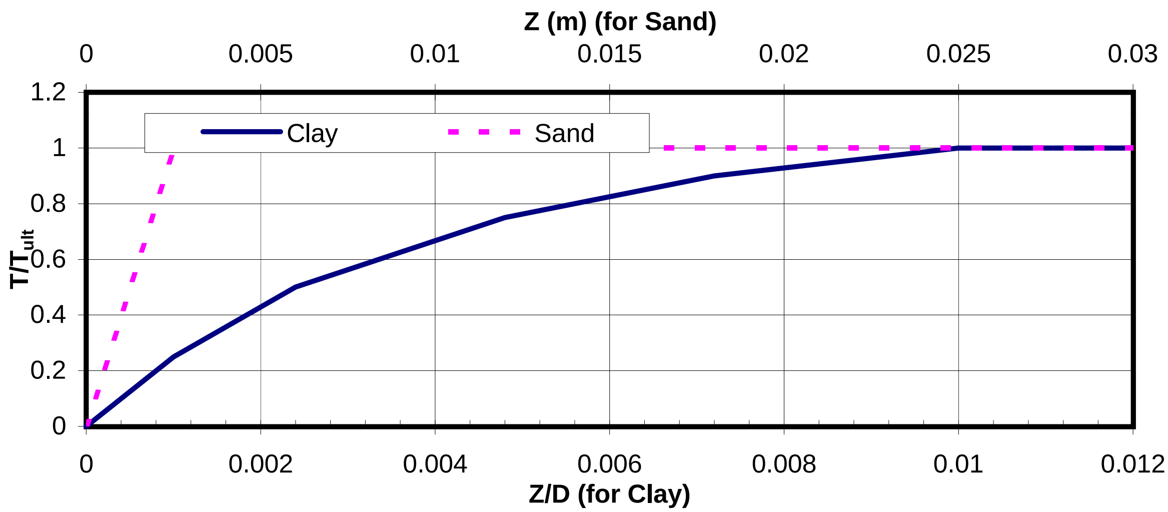
EXAMPLE AXIAL PILE LOAD TRANSFER-DISPLACEMENT CURVES
Skyway Temporary Tower: CE and CW, Pier E16 to E17
0.61-Meter-Diameter Steel Pipe Pile (Pile Tip Depth = 26.8 meters)
SFOBB East Span Seismic Safety Project



Subsurface profile is based on Boring 98-39 and should be refined to reflect location-specific subsurface exploration.

Depth (m)	Soil Type	T_{ult} (kPa)
0.0	Clay	0.0
3.7	Clay	9.1
3.7	Clay	11.5
6.1	Clay	16.8
8.8	Clay	21.5
8.8	Sand	29.7
13.3	Sand	56.0
13.3	Clay	48.8
17.7	Clay	62.2
17.7	Clay	114.9
19.8	Clay	81.9
21.9	Clay	87.1
21.9	Clay	122.1
26.2	Clay	82.4
29.6	Clay	93.8

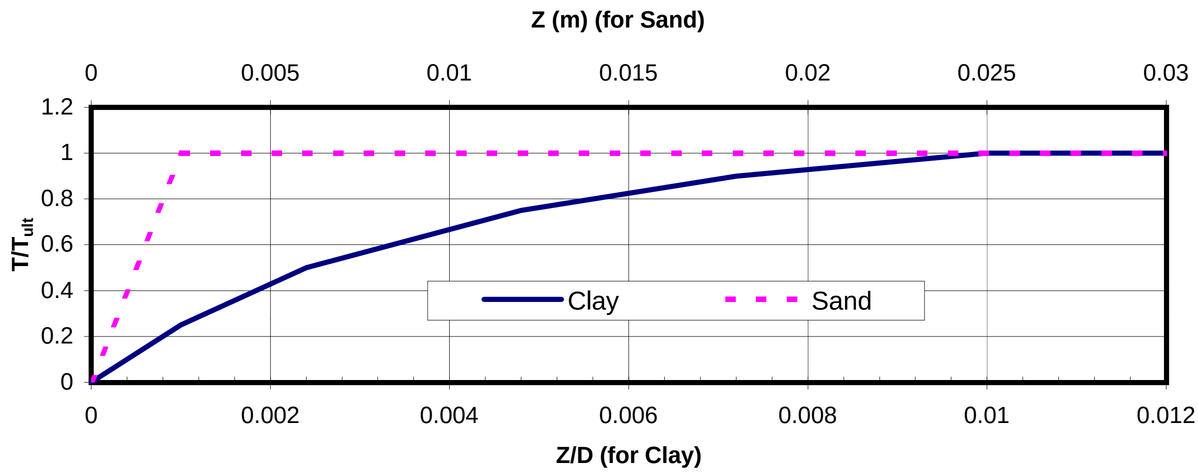
EXAMPLE AXIAL PILE LOAD TRANSFER-DISPLACEMENT CURVES
Skyway Temporary Tower: CE and CW, Pier E16 to E17
0.41-Meter-Square Precast Concrete Pile (Pile Tip Depth = 29.6 meters)
SFOBB East Span Seismic Safety Project



Subsurface profile is based on Boring 98-39 and should be refined to reflect location-specific subsurface exploration.

Depth (m)	Soil Type	T_{ult} (kPa)
0.0	Clay	0.0
3.7	Clay	9.1
3.7	Clay	11.5
6.1	Clay	16.8
8.8	Clay	21.5
8.8	Sand	29.7
13.3	Sand	56.0
13.3	Clay	48.8
17.7	Clay	62.2
17.7	Clay	114.9
19.8	Clay	81.9
21.9	Clay	87.1
21.9	Clay	122.1
23.5	Clay	105.3

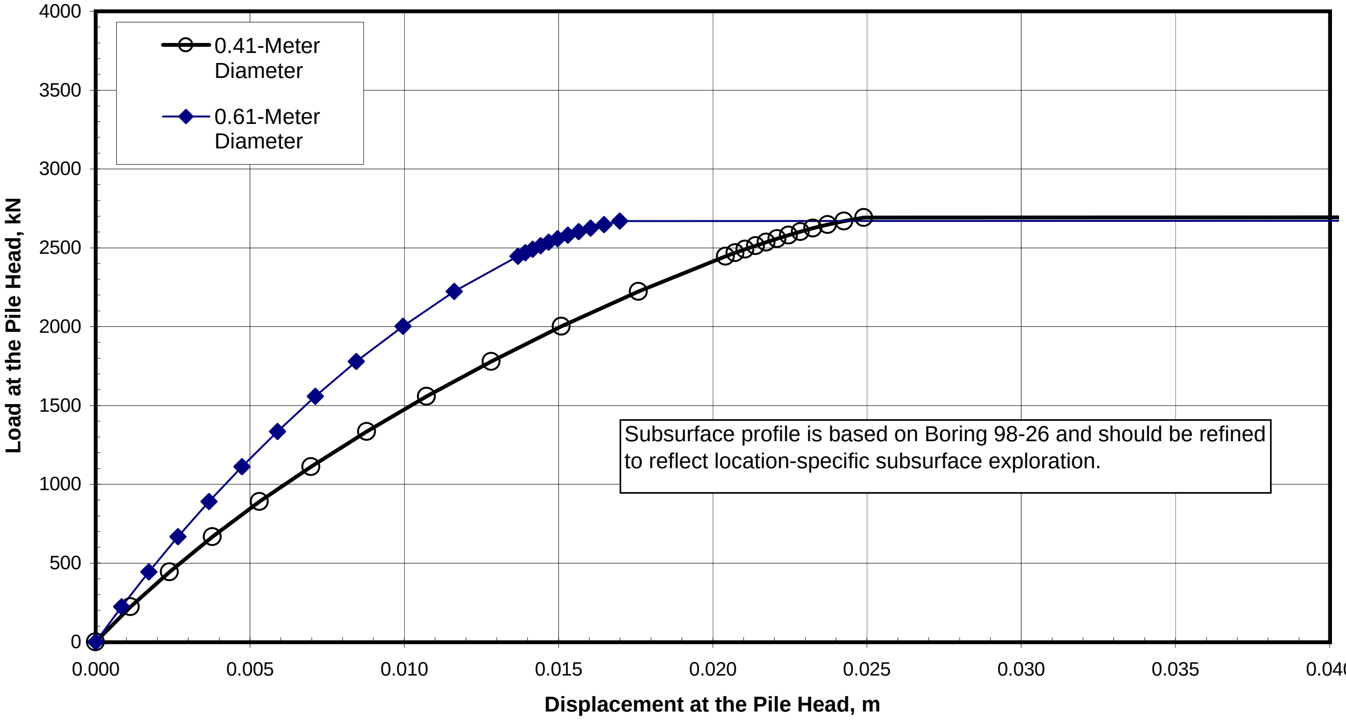
EXAMPLE AXIAL PILE LOAD TRANSFER-DISPLACEMENT CURVES
Skyway Temporary Tower: CE and CW, Pier E16 to E17
0.61-Meter-Square Precast Concrete Pile (Pile Tip Depth = 23.5 meters)
SFOBB East Span Seismic Safety Project



Subsurface profile is based on Boring 98-39 and should be refined to reflect location-specific subsurface exploration.

Depth (m)	Soil Type	T_{ult} (kPa)
0.0	Clay	0.0
3.7	Clay	9.1
3.7	Clay	11.5
6.1	Clay	16.8
8.8	Clay	21.5
8.8	Sand	29.7
13.3	Sand	56.0
13.3	Clay	48.8
17.7	Clay	62.2
17.7	Clay	114.9
19.8	Clay	81.9
21.9	Clay	87.1
21.9	Clay	122.1
26.2	Clay	82.4
31.1	Clay	99.1
31.1	Clay	113.0
31.4	Clay	113.5

EXAMPLE AXIAL PILE LOAD TRANSFER-DISPLACEMENT CURVES
Skyway Temporary Tower: CE and CW, Pier E16 to E17
0.36-Meter Steel H Pile (Pile Tip Depth = 31.4 meters)
SFOBB East Span Seismic Safety Project



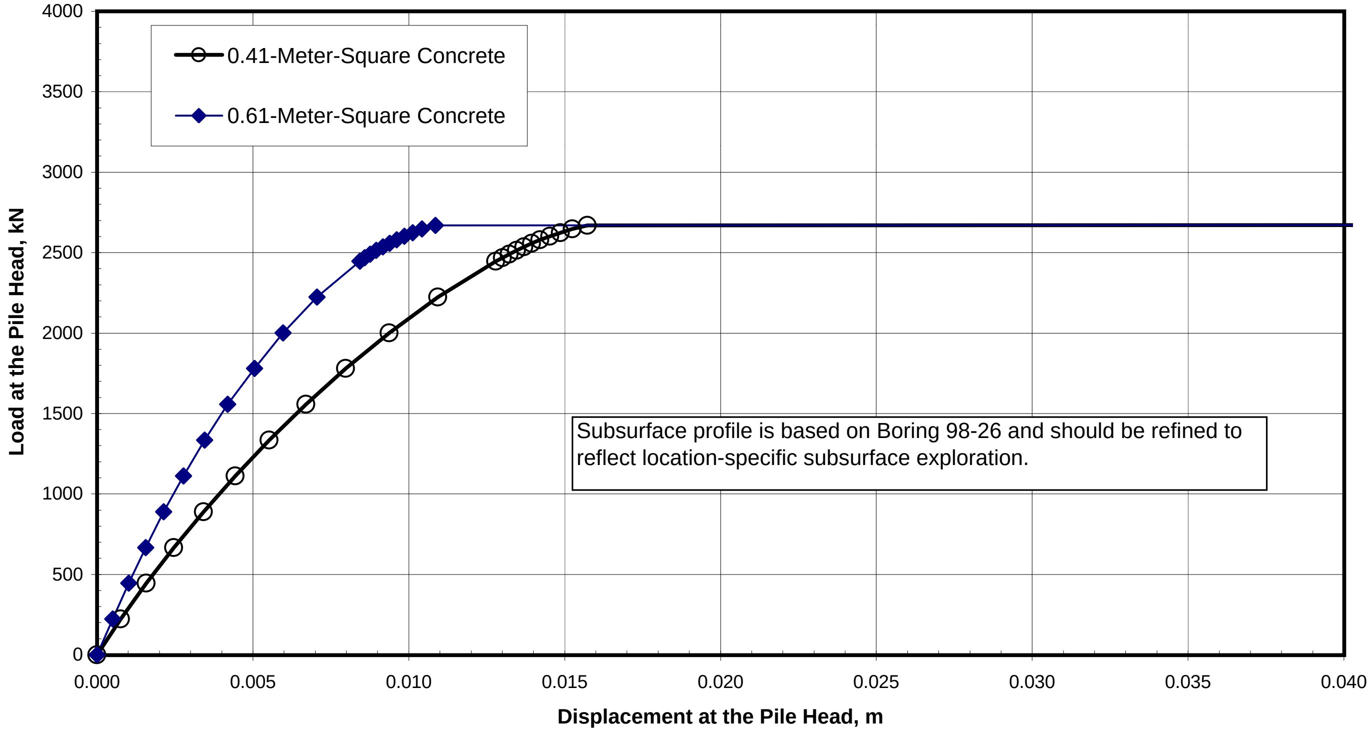
Axial Load at Pile Head (kN)	Pile Head Displacement (m) 0.41-Meter Diameter	Pile Head Displacement (m) 0.61-Meter Diameter
0	0.000	0.000
222	0.001	0.001
445	0.002	0.002
667	0.004	0.003
890	0.005	0.004
1112	0.007	0.005
1334	0.009	0.006
1557	0.011	0.007
1779	0.013	0.008
2002	0.015	0.010
2224	0.018	0.012
2447	0.020	0.014
2469	0.021	0.014

Axial Load at Pile Head (kN)	Pile Head Displacement (m) 0.41-Meter Diameter	Pile Head Displacement (m) 0.61-Meter Diameter
2491	0.021	0.014
2513	0.021	0.014
2535	0.022	0.015
2558	0.022	0.015
2580	0.022	0.015
2602	0.023	0.016
2624	0.023	0.016
2647	0.024	0.016
2669	0.024	0.017
2691	0.025	1.480
2713	0.756	1.480
2736	0.756	1.480
2758	0.756	1.480

Note:
Pile Head Load-Displacement curves are based on a uniform 12.7 mm wall thickness and static loading conditions.

0.61-Meter-Diameter Pile, Tip Depth: 30.8 meters below seafloor
0.41-Meter-Diameter Pile, Tip Depth: 38.7 meters below seafloor

EXAMPLE STATIC AXIAL PILE HEAD LOAD-DEFORMATION CURVES (TENSION)
Skyway Temporary Tower: AE and AW, Pier E2 to E3
0.41- and 0.61-Meter-Diameter Steel Pipe Piles
SFOBB East Span Seismic Safety Project



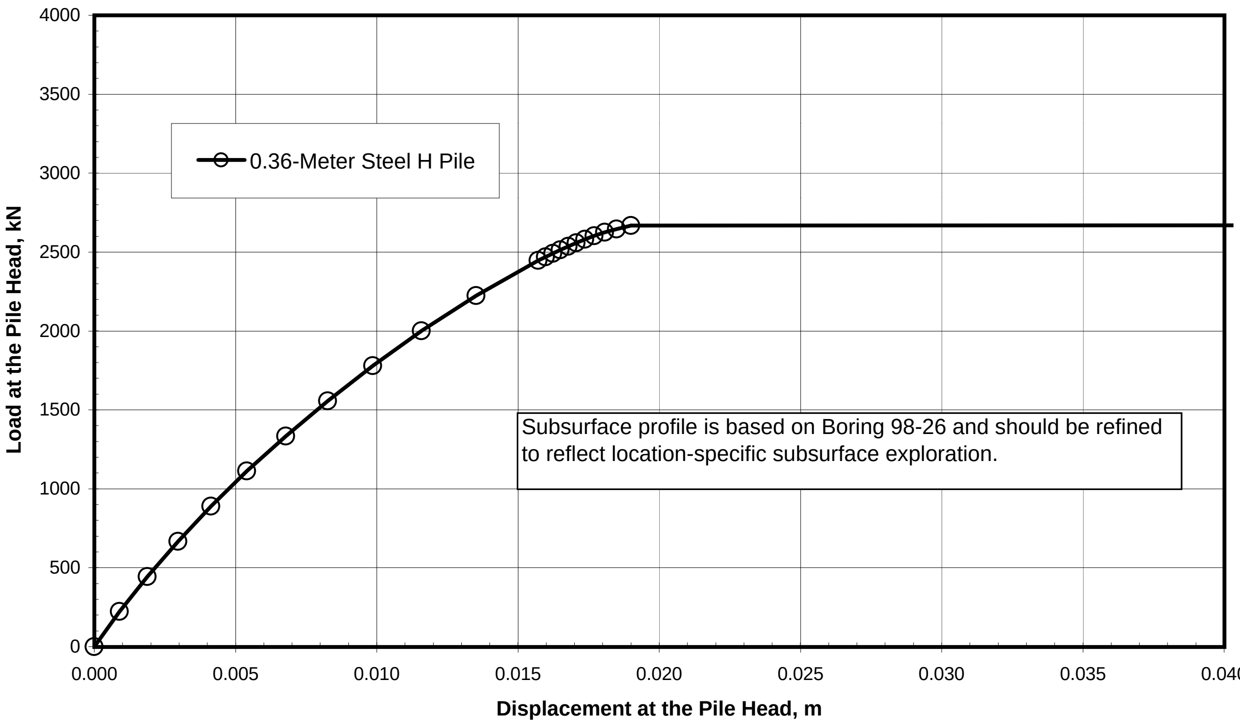
Axial Load at Pile Head (kN)	Pile Head Displacement (m), 0.41-Meter Width	Pile Head Displacement (m), 0.61-Meter Width
0	0.000	0.000
222	0.001	0.001
445	0.002	0.001
667	0.002	0.002
890	0.003	0.002
1112	0.004	0.003
1334	0.006	0.003
1557	0.007	0.004
1779	0.008	0.005
2002	0.009	0.006
2224	0.011	0.007
2447	0.013	0.008
2469	0.013	0.009

Axial Load at Pile Head (kN)	Pile Head Displacement (m), 0.41-Meter Width	Pile Head Displacement (m), 0.61-Meter Width
2491	0.013	0.009
2513	0.013	0.009
2535	0.014	0.009
2558	0.014	0.009
2580	0.014	0.010
2602	0.015	0.010
2624	0.015	0.010
2647	0.015	0.010
2669	0.016	0.011
2691	0.747	1.474
2713	0.747	1.474
2736	0.747	1.474
2758	0.747	1.474

Note:
Pile Head Load-Displacement curves are based on static loading conditions.

0.61-Meter-Square Concrete Pile, Tip Depth: 27.4 meters below seafloor
0.41-Meter-Square Concrete Pile, Tip Depth: 33.5 meters below seafloor

EXAMPLE STATIC AXIAL PILE HEAD LOAD-DEFORMATION CURVES (TENSION)
Skyway Temporary Tower: AE and AW, Pier E2 to E3
0.41- and 0.61-Meter-Square Precast Concrete Piles
SFOBB East Span Seismic Safety Project



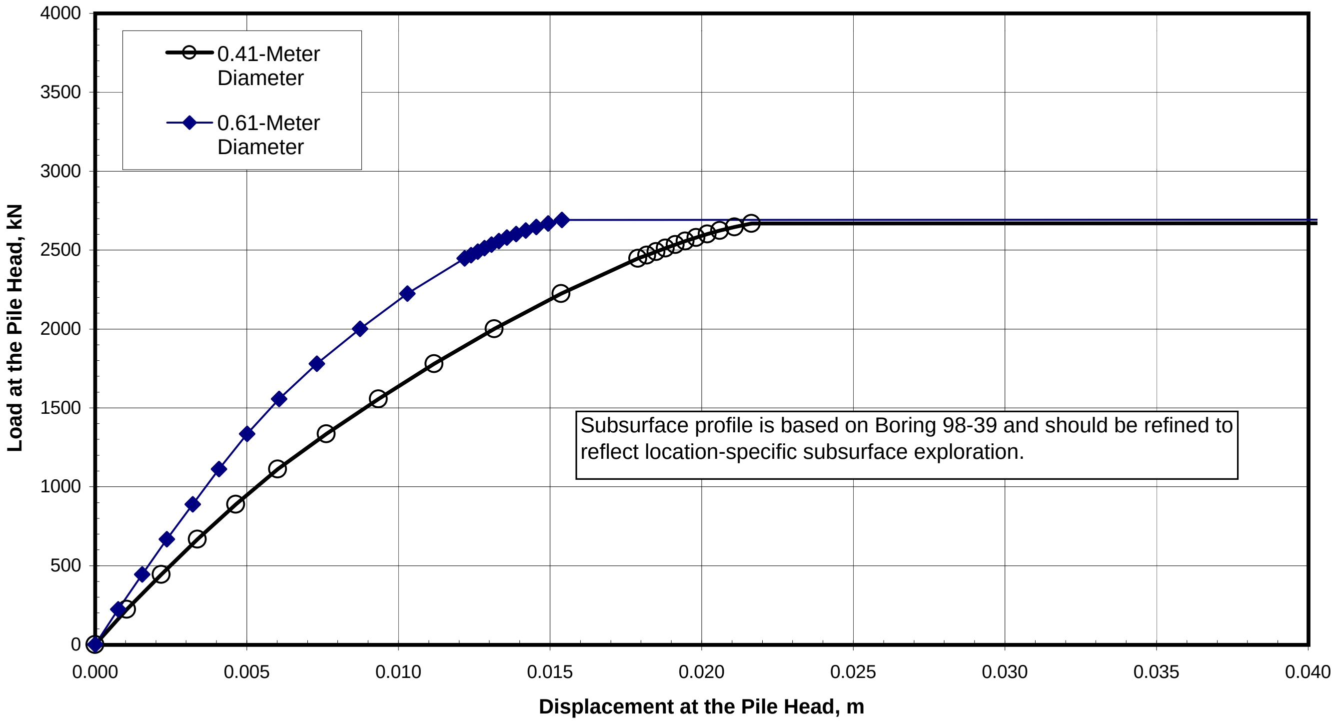
Axial Load at Pile Head (kN)	Pile Head Displacement (m), 0.36-Meter Steel H Pile
0	0.000
222	0.001
445	0.002
667	0.003
890	0.004
1112	0.005
1334	0.007
1557	0.008
1779	0.010
2002	0.012
2224	0.014
2447	0.016
2469	0.016

Axial Load at Pile Head (kN)	Pile Head Displacement (m), 0.36-Meter Steel H Pile
2491	0.016
2513	0.017
2535	0.017
2558	0.017
2580	0.017
2602	0.018
2624	0.018
2647	0.018
2669	0.019
2691	0.751
2713	0.751
2736	0.751
2758	0.751

Note:
Pile Head Load-Displacement curves are based on static loading conditions.

0.36-Meter Steel H Pile, Tip Depth: 35.4 meters below seafloor

EXAMPLE STATIC AXIAL PILE HEAD LOAD-DEFORMATION CURVES (TENSION)
Skyway Temporary Tower: AE and AW, Pier E2 to E3
0.36-Meter Steel H Pile
SFOBB East Span Seismic Safety Project



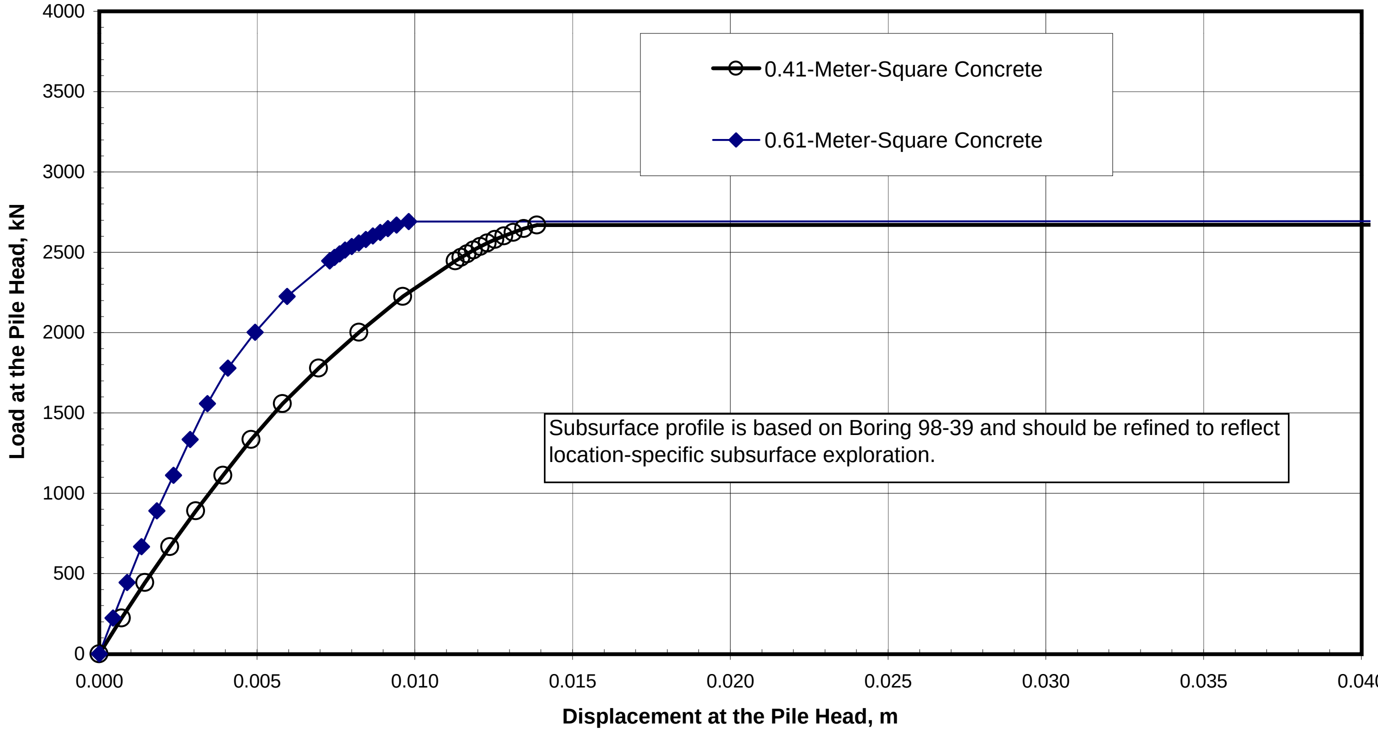
Axial Load at Pile Head (kN)	Pile Head Displacement (m), 0.41-Meter Diameter	Pile Head Displacement (m), 0.61-Meter Diameter
0	0.000	0.000
222	0.001	0.001
445	0.002	0.002
667	0.003	0.002
890	0.005	0.003
1112	0.006	0.004
1334	0.008	0.005
1557	0.009	0.006
1779	0.011	0.007
2002	0.013	0.009
2224	0.015	0.010
2447	0.018	0.012
2469	0.018	0.012

Axial Load at Pile Head (kN)	Pile Head Displacement (m), 0.41-Meter Diameter	Pile Head Displacement (m), 0.61-Meter Diameter
2491	0.018	0.013
2513	0.019	0.013
2535	0.019	0.013
2558	0.019	0.013
2580	0.020	0.014
2602	0.020	0.014
2624	0.021	0.014
2647	0.021	0.015
2669	0.022	0.015
2691	0.753	0.015
2713	0.753	1.478
2736	0.753	1.478
2758	0.753	1.478

Note:
Pile Head Load-Displacement curves are based on a uniform 12.7 mm wall thickness and static loading conditions.

0.61-Meter-Diameter Pile, Tip Depth: 26.8 meters below seafloor
0.41-Meter-Diameter Pile, Tip Depth: 34.1 meters below seafloor

EXAMPLE STATIC AXIAL PILE HEAD LOAD-DEFORMATION CURVES (TENSION)
Skyway Temporary Tower: CE and CW, Pier E16 to E17
0.41- and 0.61-Meter-Diameter Steel Pipe Piles
SFOBB East Span Seismic Safety Project

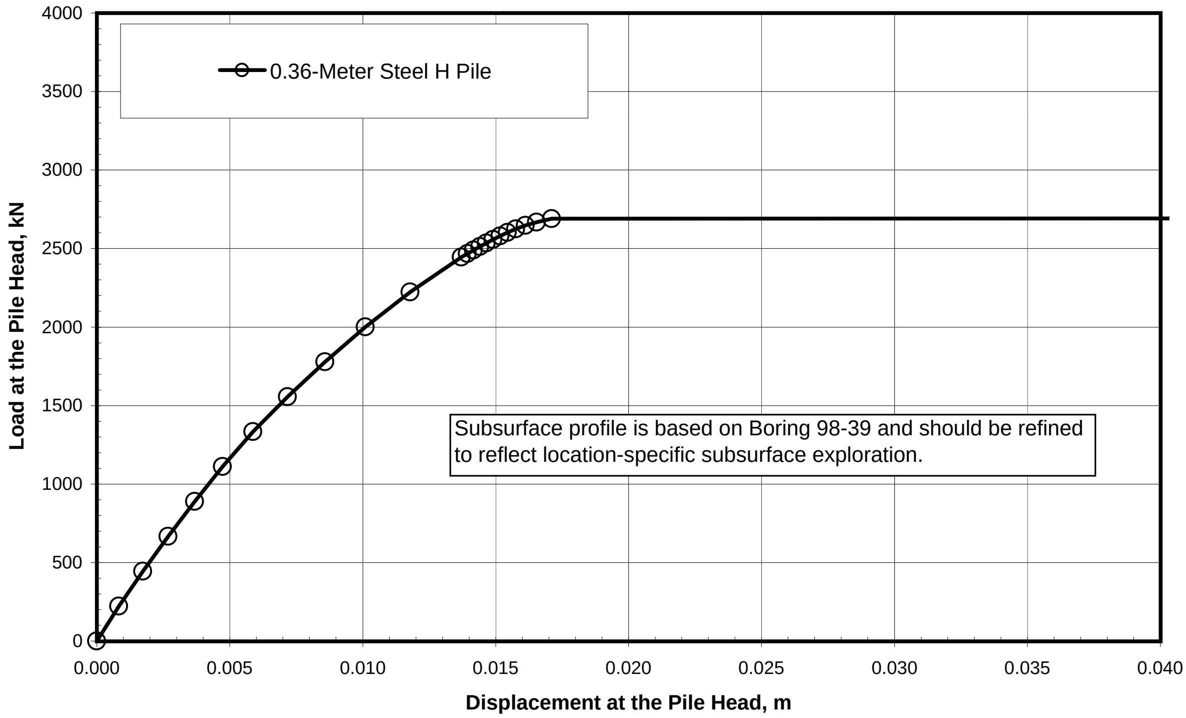


Axial Load at Pile Head (kN)	Pile Head Displacement (m), 0.41-Meter Width	Pile Head Displacement (m) 0.61-Meter Width
0	0.000	0.000
222	0.001	0.000
445	0.001	0.001
667	0.002	0.001
890	0.003	0.002
1112	0.004	0.002
1334	0.005	0.003
1557	0.006	0.003
1779	0.007	0.004
2002	0.008	0.005
2224	0.010	0.006
2447	0.011	0.007
2469	0.011	0.007
2491	0.012	0.008
2513	0.012	0.008
2535	0.012	0.008
2558	0.012	0.008
2580	0.013	0.008
2602	0.013	0.009
2624	0.013	0.009
2647	0.013	0.009
2669	0.014	0.009
2691	0.745	0.010
2713	0.745	1.473
2736	0.745	1.473
2758	0.745	1.473

Note:
Pile Head Load-Displacement curves are based on static loading conditions.

0.61-Meter-Square Concrete Pile, Tip Depth: 23.5 meters below seafloor
0.41-Meter-Square Concrete Pile, Tip Depth: 29.6 meters below seafloor

EXAMPLE STATIC AXIAL PILE HEAD LOAD-DEFORMATION CURVES (TENSION)
Skyway Temporary Tower: CE and CW, Pier E16 to E17
0.41- and 0.61-Meter-Square Precast Concrete Piles
SFOBB East Span Seismic Safety Project



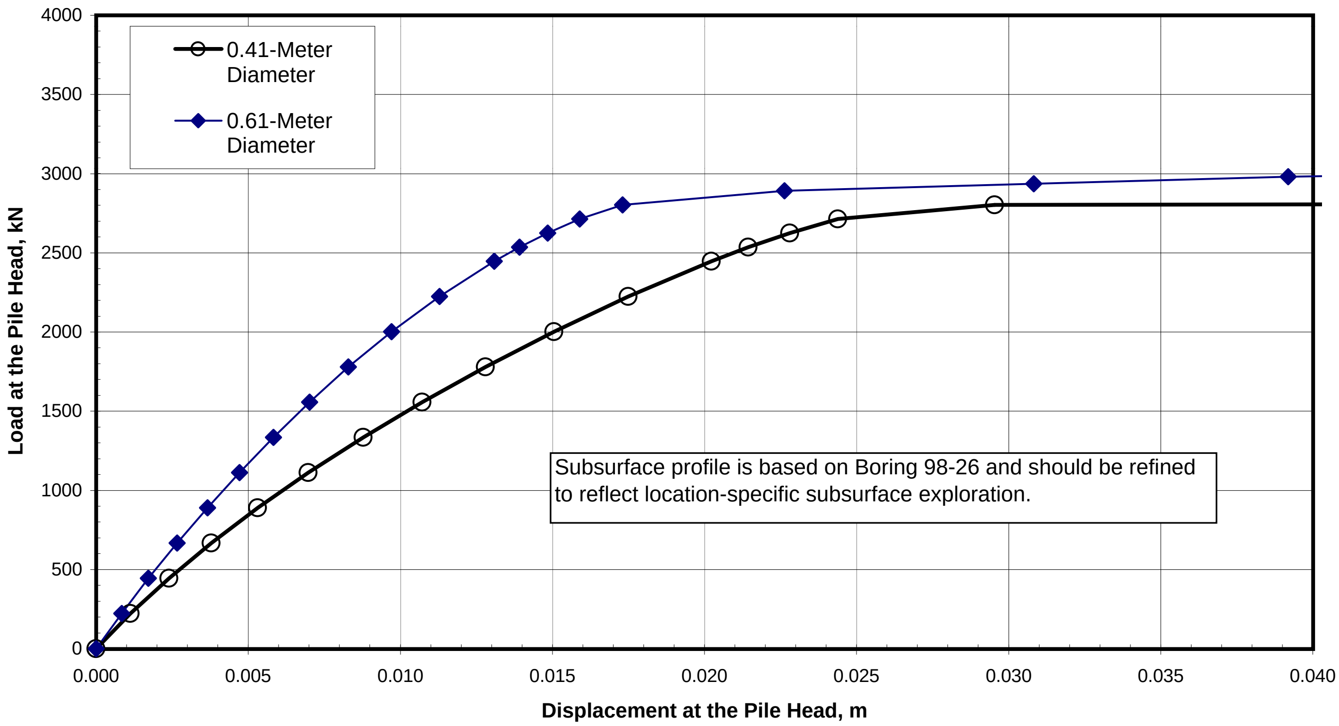
Axial Load at Pile Head (kN)	Pile Head Displacement (m), 0.36-Meter Steel H Pile
0	0.000
222	0.001
445	0.002
667	0.003
890	0.004
1112	0.005
1334	0.006
1557	0.007
1779	0.009
2002	0.010
2224	0.012
2447	0.014
2469	0.014

Axial Load at Pile Head (kN)	Pile Head Displacement (m), 0.36-Meter Steel H Pile
2491	0.014
2513	0.014
2535	0.015
2558	0.015
2580	0.015
2602	0.015
2624	0.016
2647	0.016
2669	0.017
2691	0.017
2713	0.749
2736	0.749
2758	0.749

Note: Pile Head Load-Displacement curves are based on static loading conditions.

0.36-Meter Steel H Pile, Tip Depth: 31.4 meters below seafloor

EXAMPLE STATIC AXIAL PILE HEAD LOAD-DEFORMATION CURVES (TENSION)
Skyway Temporary Tower: CE and CW, Pier E16 to E17
0.36-Meter Steel H Pile
SFOBB East Span Seismic Safety Project



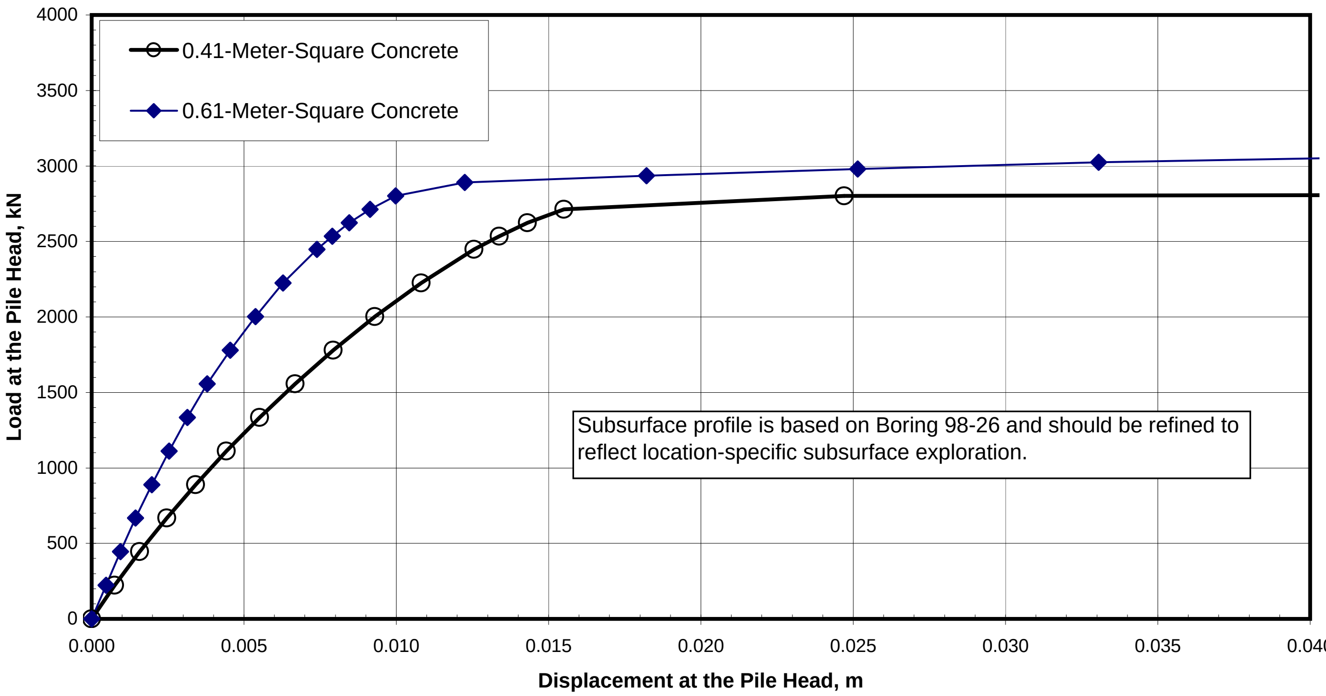
Axial Load at Pile Head (kN)	Pile Head Displacement (m) 0.41-Meter Diameter	Pile Head Displacement (m) 0.61-Meter Diameter
0	0.000	0.000
222	0.001	0.001
445	0.002	0.002
667	0.004	0.003
890	0.005	0.004
1112	0.007	0.005
1334	0.009	0.006
1557	0.011	0.007
1779	0.013	0.008
2002	0.015	0.010
2224	0.017	0.011
2447	0.020	0.013
2535	0.021	0.014

Axial Load at Pile Head (kN)	Pile Head Displacement (m) 0.41-Meter Diameter	Pile Head Displacement (m) 0.61-Meter Diameter
2624	0.023	0.015
2713	0.024	0.016
2802	0.030	0.017
2891	4.009	0.023
2936	4.931	0.031
2980	5.853	0.039
3025	6.775	0.054
3069	7.697	0.071
3114	8.619	6.653
3158	9.541	7.336
3203	10.463	8.019
3247	11.385	8.702
3292	12.307	9.385

Note:
Pile Head Load-Displacement curves are based on a uniform 12.7 mm wall thickness and static loading conditions.

0.61-Meter-Diameter Pile, Tip Depth: 30.8 meters below seafloor
0.41-Meter-Diameter Pile, Tip Depth: 38.7 meters below seafloor

EXAMPLE STATIC AXIAL PILE HEAD LOAD-DEFORMATION CURVES (COMPRESSION)
Skyway Temporary Tower: AE and AW, Pier E2 to E3
0.41- and 0.61-Meter-Diameter Steel Pipe Piles
SFOBB East Span Seismic Safety Project



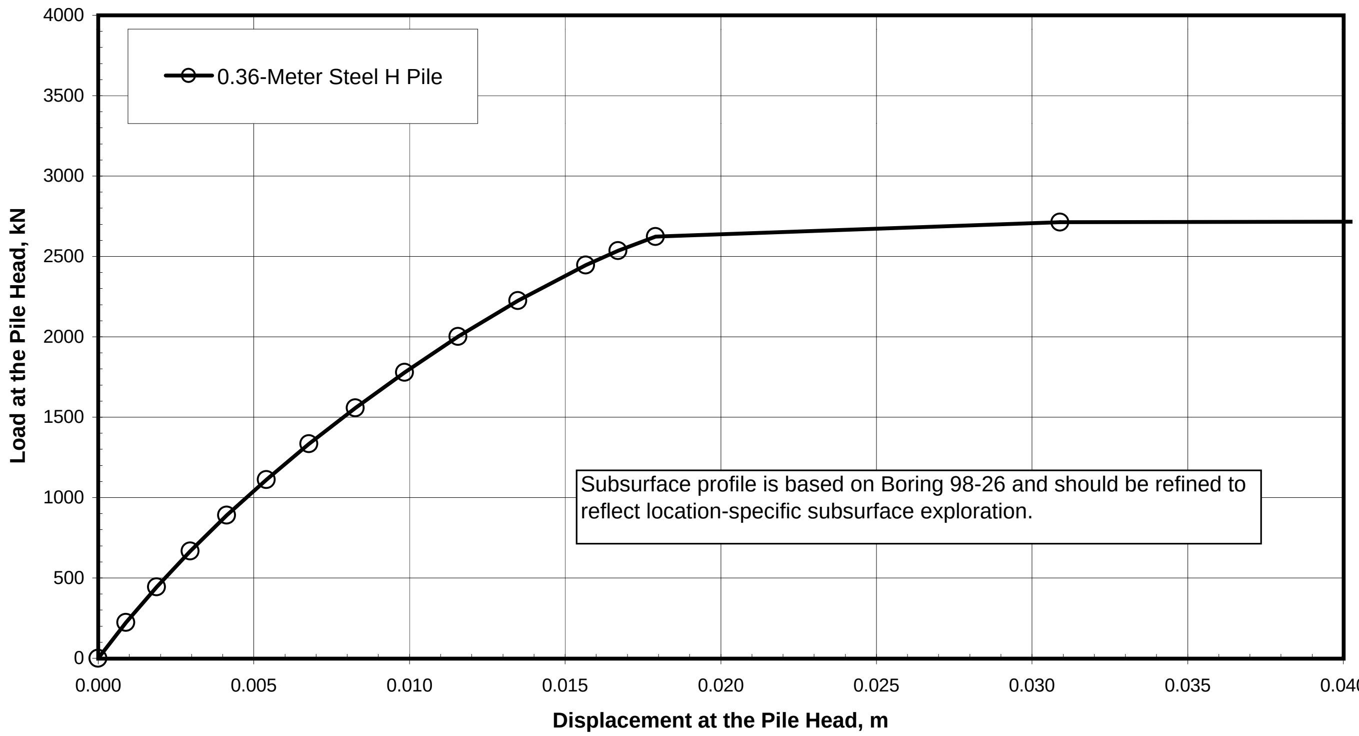
Axial Load at Pile Head (kN)	Pile Head Displacement (m) 0.41-Meter Width	Pile Head Displacement (m) 0.61-Meter Width
0	0.000	0.000
222	0.001	0.000
445	0.002	0.001
667	0.002	0.001
890	0.003	0.002
1112	0.004	0.003
1334	0.006	0.003
1557	0.007	0.004
1779	0.008	0.005
2002	0.009	0.005
2224	0.011	0.006
2447	0.013	0.007
2535	0.013	0.008

Axial Load at Pile Head (kN)	Pile Head Displacement (m) 0.41-Meter Width	Pile Head Displacement (m) 0.61-Meter Width
2624	0.014	0.008
2713	0.016	0.009
2802	0.025	0.010
2891	4.339	0.012
2936	5.220	0.018
2980	6.101	0.025
3025	6.982	0.033
3069	7.862	0.045
3114	8.743	0.060
3158	9.624	6.452
3203	10.505	7.041
3247	11.386	7.630
3292	12.266	8.219

Note:
Pile Head Load-Displacement curves are based on static loading conditions.

0.61-Meter-Square Concrete Pile, Tip Depth: 27.4 meters below seafloor
0.41-Meter-Square Concrete Pile, Tip Depth: 33.5 meters below seafloor

EXAMPLE STATIC AXIAL PILE HEAD LOAD-DEFORMATION CURVES (COMPRESSION)
Skyway Temporary Tower: AE and AW, Pier E2 to E3
0.41- and 0.61-Meter-Square Precast Concrete Piles
SFOBB East Span Seismic Safety Project



Axial Load at Pile Head (kN)	Pile Head Displacement (m) 0.36-Meter Steel H Pile	Axial Load at Pile Head (kN)	Pile Head Displacement (m) 0.36-Meter Steel H Pile
0	0.000	2624	0.018
222	0.001	2713	0.031
445	0.002	2802	9.142
667	0.003	2891	15.608
890	0.004	2936	18.841
1112	0.005	2980	22.074
1334	0.007	3025	25.307
1557	0.008	3069	28.540
1779	0.010	3114	30.480
2002	0.012	3158	30.480
2224	0.013	3203	30.480
2447	0.016	3247	30.480
2535	0.017	3292	30.480

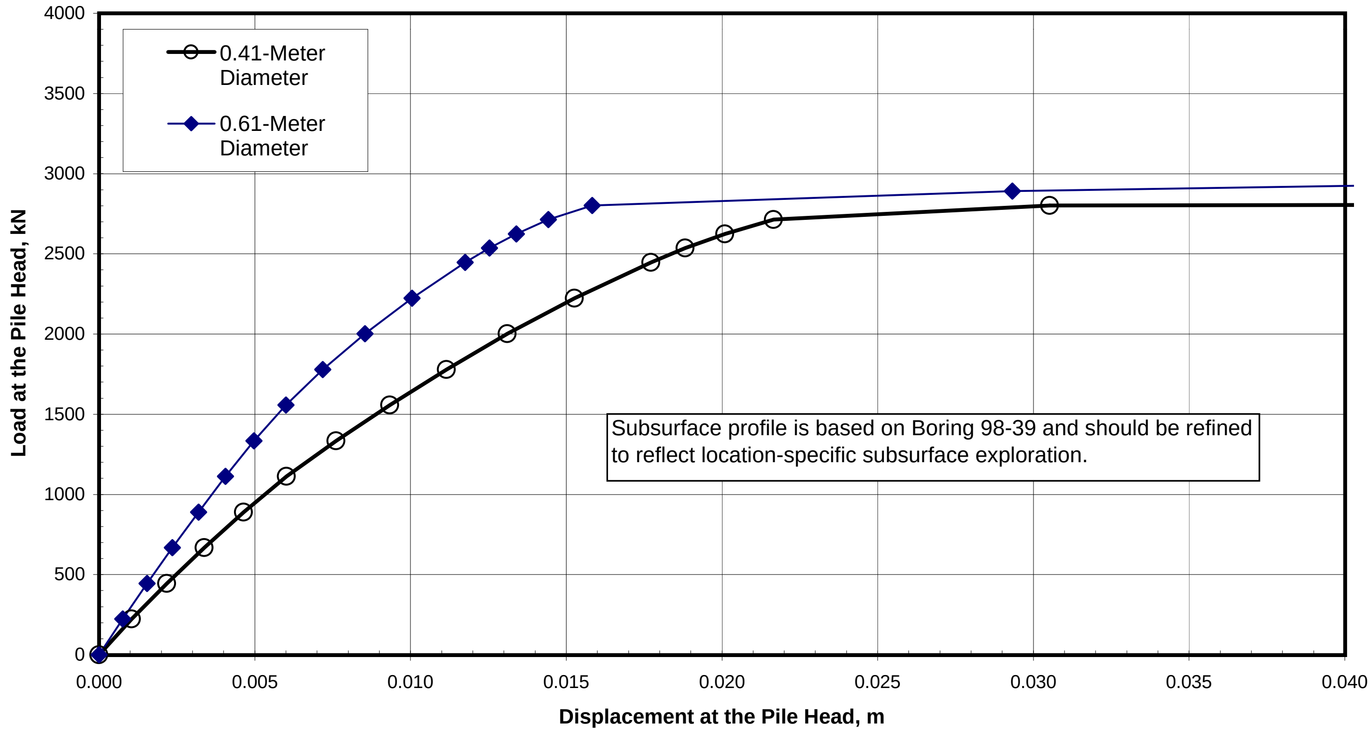
Note:
Pile Head Load-Displacement curves are based on static loading conditions.

0.36-Meter Steel H Pile, Tip Depth: 35.4 meters below seafloor

EXAMPLE STATIC AXIAL PILE HEAD LOAD-DEFORMATION CURVES (COMPRESSION)
Skyway Temporary Tower: AE and AW, Pier E2 to E3
0.36-Meter Steel H Pile
SFOBB East Span Seismic Safety Project

EXAMPLE CALCULATIONS
NOT FOR DESIGN

Earth Mechanics



Axial Load at Pile Head (kN)	Pile Head Displacement (m) 0.41-Meter Diameter	Pile Head Displacement (m) 0.61-Meter Diameter
0	0.000	0.000
222	0.001	0.001
445	0.002	0.002
667	0.003	0.002
890	0.005	0.003
1112	0.006	0.004
1334	0.008	0.005
1557	0.009	0.006
1779	0.011	0.007
2002	0.013	0.009
2224	0.015	0.010
2447	0.018	0.012
2535	0.019	0.013

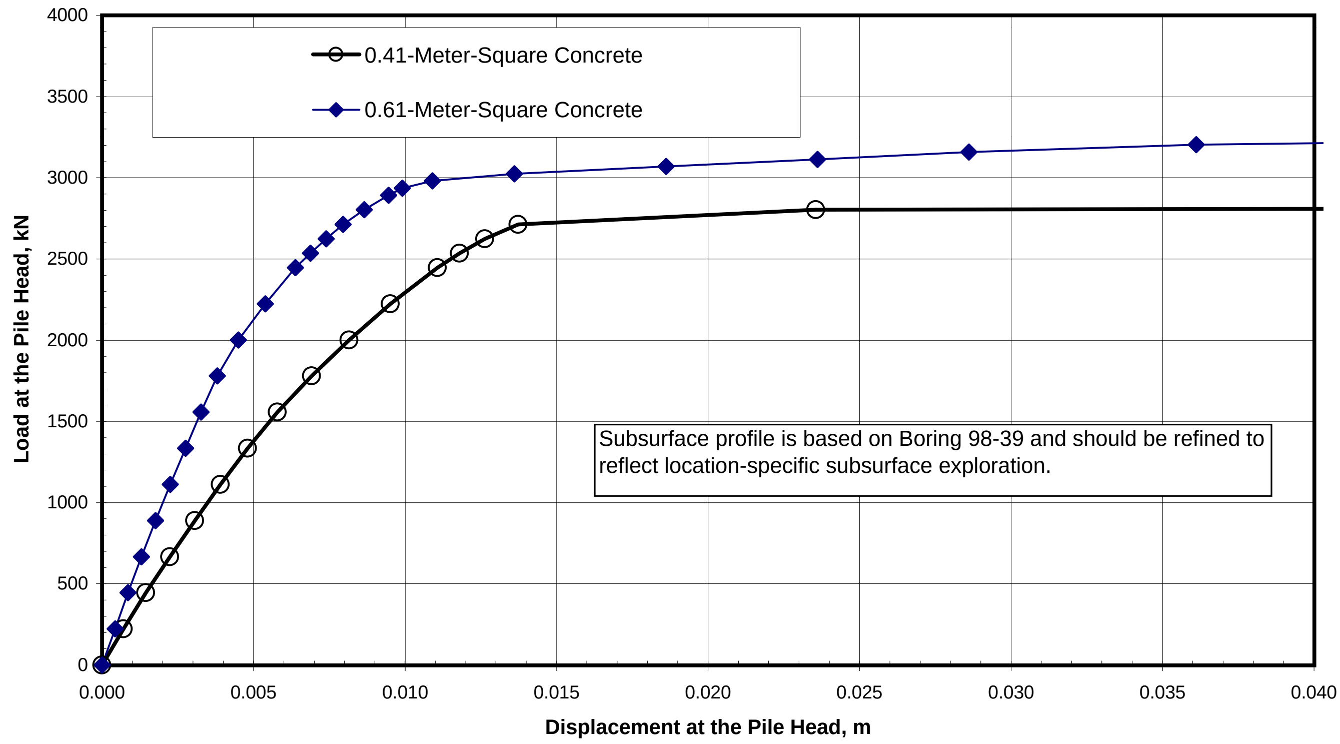
Axial Load at Pile Head (kN)	Pile Head Displacement (m) 0.41-Meter Diameter	Pile Head Displacement (m) 0.61-Meter Diameter
2624	0.020	0.013
2713	0.022	0.014
2802	0.031	0.016
2891	4.398	0.029
2936	5.342	0.044
2980	6.285	0.066
3025	7.229	6.841
3069	8.173	7.797
3114	9.117	8.752
3158	10.061	9.708
3203	11.005	10.664
3247	11.949	11.620
3292	12.893	12.575

Note:
Pile Head Load-Displacement curves are based on a uniform 12.7 mm wall thickness and static loading conditions.

0.61-Meter-Diameter Pile, Tip Depth: 26.8 meters below seafloor
0.41-Meter-Diameter Pile, Tip Depth: 34.1 meters below seafloor

EXAMPLE STATIC AXIAL PILE HEAD LOAD-DEFORMATION CURVES (COMPRESSION)
Skyway Temporary Tower: CE and CW, Pier E16 to E17
0.41- and 0.61-Meter-Diameter Steel Pipe Piles
SFOBB East Span Seismic Safety Project

EXAMPLE CALCULATIONS
NOT FOR DESIGN



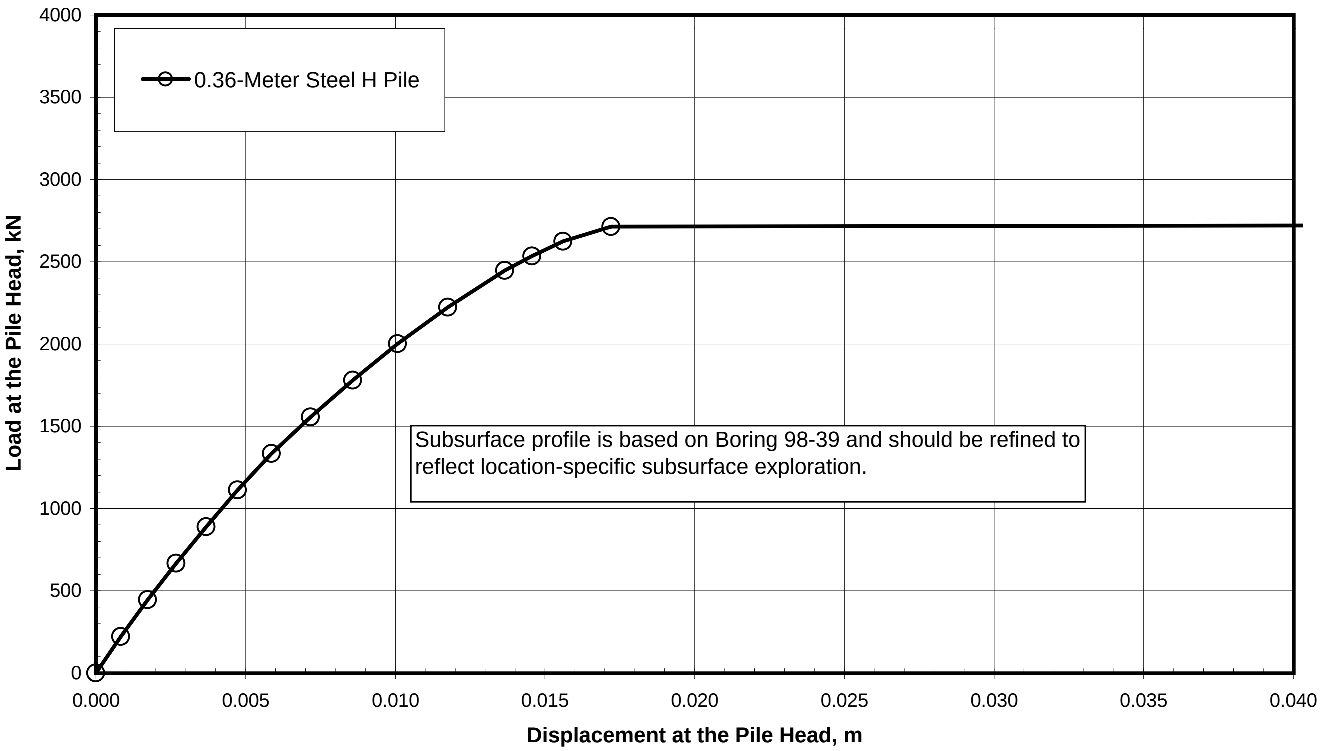
Axial Load at Pile Head (kN)	Pile Head Displacement (m) 0.41-Meter Width	Pile Head Displacement (m) 0.61-Meter Width
0	0.000	0.000
222	0.001	0.000
445	0.001	0.001
667	0.002	0.001
890	0.003	0.002
1112	0.004	0.002
1334	0.005	0.003
1557	0.006	0.003
1779	0.007	0.004
2002	0.008	0.005
2224	0.010	0.005
2447	0.011	0.006
2535	0.012	0.007

Axial Load at Pile Head (kN)	Pile Head Displacement (m) 0.41-Meter Width	Pile Head Displacement (m) 0.61-Meter Width
2624	0.013	0.007
2713	0.014	0.008
2802	0.024	0.009
2891	4.545	0.009
2936	5.500	0.010
2980	6.455	0.011
3025	7.410	0.014
3069	8.365	0.019
3114	9.320	0.024
3158	10.275	0.029
3203	11.230	0.036
3247	12.185	0.055
3292	13.140	6.122

Note:
Pile Head Load-Displacement curves are based on static loading conditions.

0.61-Meter-Square Concrete Pile, Tip Depth: 23.5 meters below seafloor
0.41-Meter-Square Concrete Pile, Tip Depth: 29.6 meters below seafloor

EXAMPLE STATIC AXIAL PILE HEAD LOAD-DEFORMATION CURVES (COMPRESSION)
Skyway Temporary Tower: CE and CW, Pier E16 to E17
0.41- and 0.61-Meter-Square Precast Concrete Piles
SFOBB East Span Seismic Safety Project



Axial Load at Pile Head (kN)	Pile Head Displacement (m) 0.36-Meter Steel H Pile
0	0.000
222	0.001
445	0.002
667	0.003
890	0.004
1112	0.005
1334	0.006
1557	0.007
1779	0.009
2002	0.010
2224	0.012
2447	0.014
2535	0.015

Axial Load at Pile Head (kN)	Pile Head Displacement (m) 0.36-Meter Steel H Pile
2624	0.016
2713	0.017
2802	6.751
2891	12.222
2936	14.958
2980	17.694
3025	20.429
3069	23.165
3114	25.901
3158	28.637
3203	30.480
3247	30.480
3292	30.480

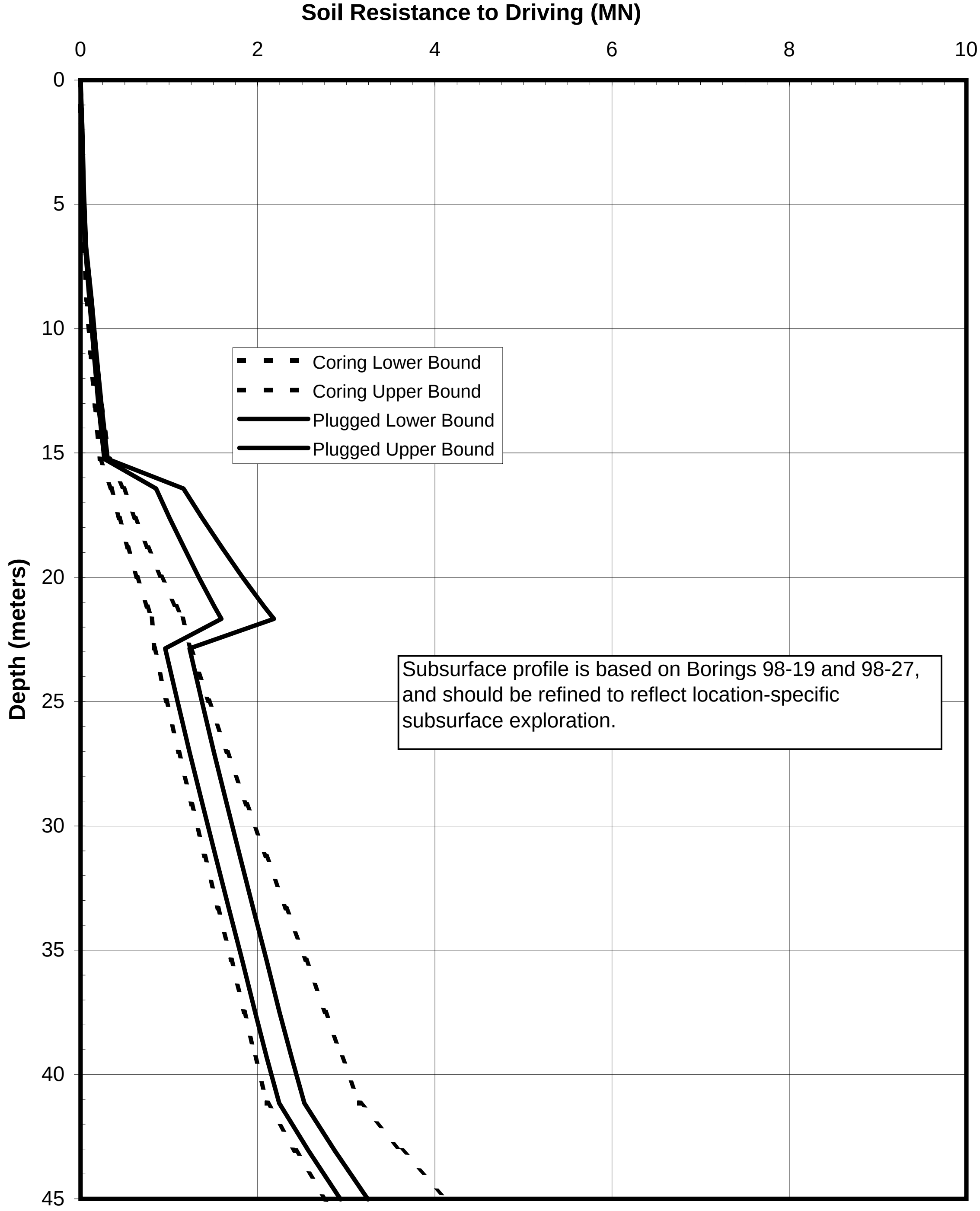
Note: Pile Head Load-Displacement curves are based on static loading conditions.

0.36-Meter Steel H Pile, Tip Depth: 31.4 meters below seafloor

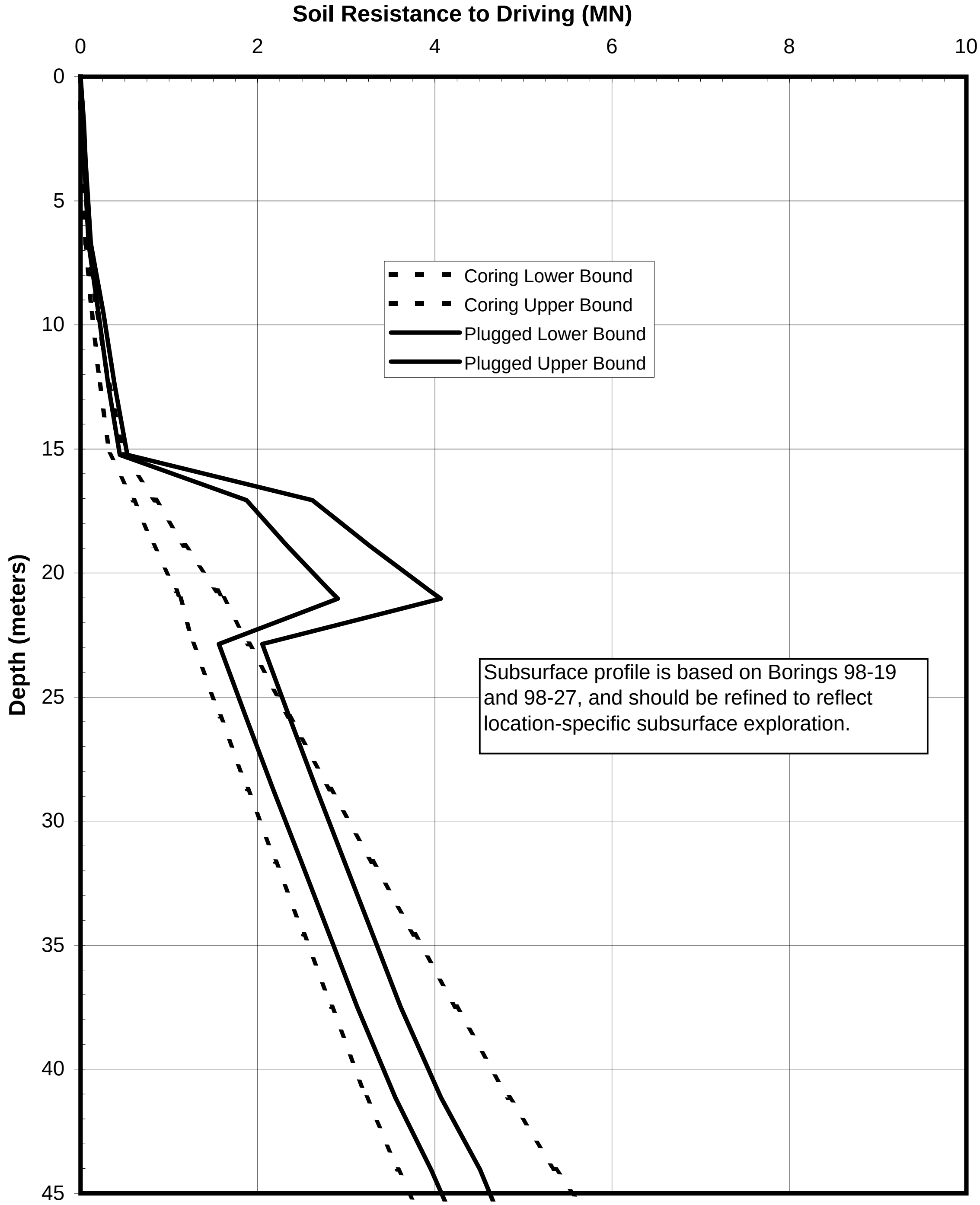
EXAMPLE STATIC AXIAL PILE HEAD LOAD-DEFORMATION CURVES (COMPRESSION)
Skyway Temporary Tower: CE and CW, Pier E16 to E17
0.36-Meter Steel H Pile
SFOBB East Span Seismic Safety Project

APPENDIX D

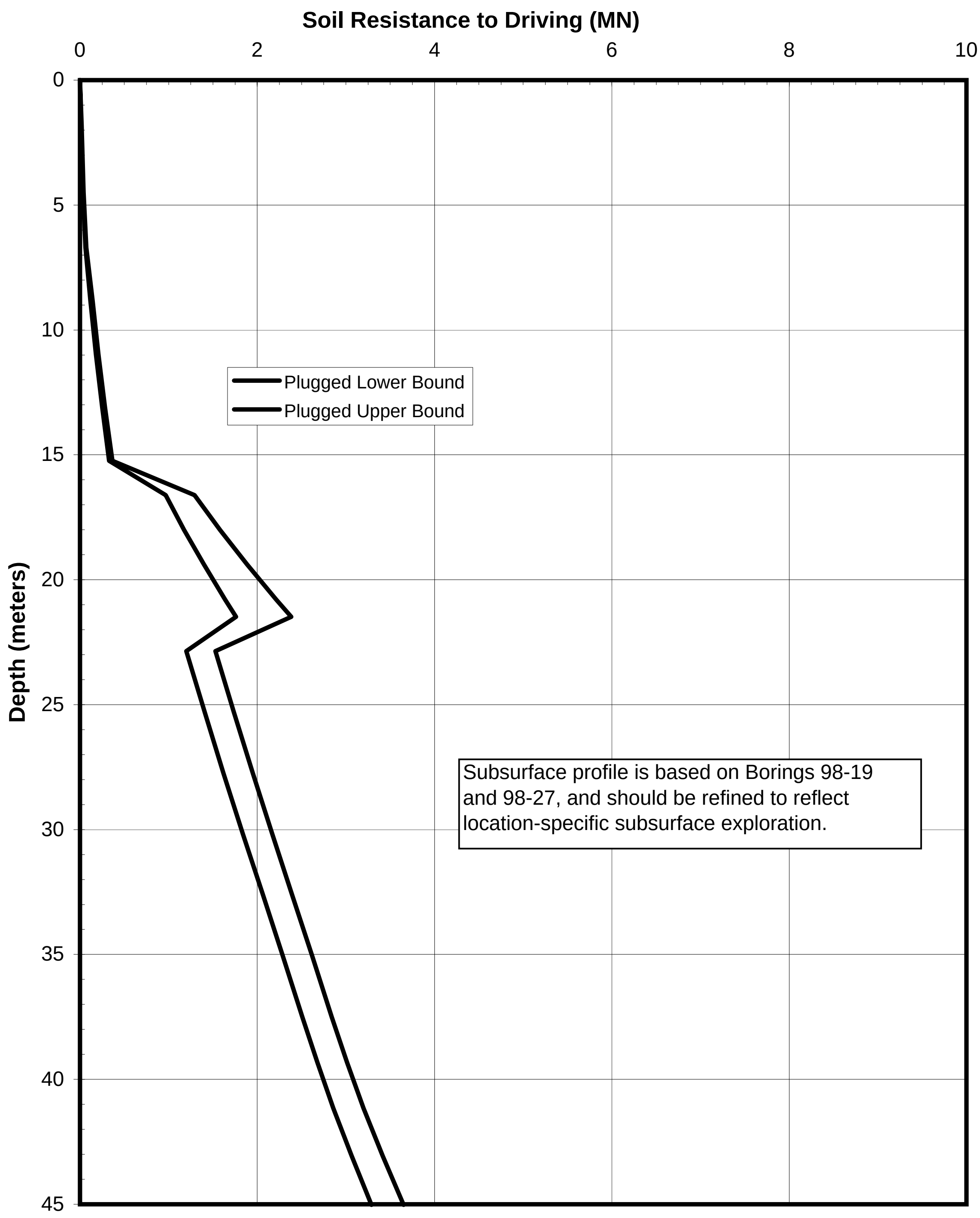
APPENDIX D
PILE DRIVABILITY ANALYSIS



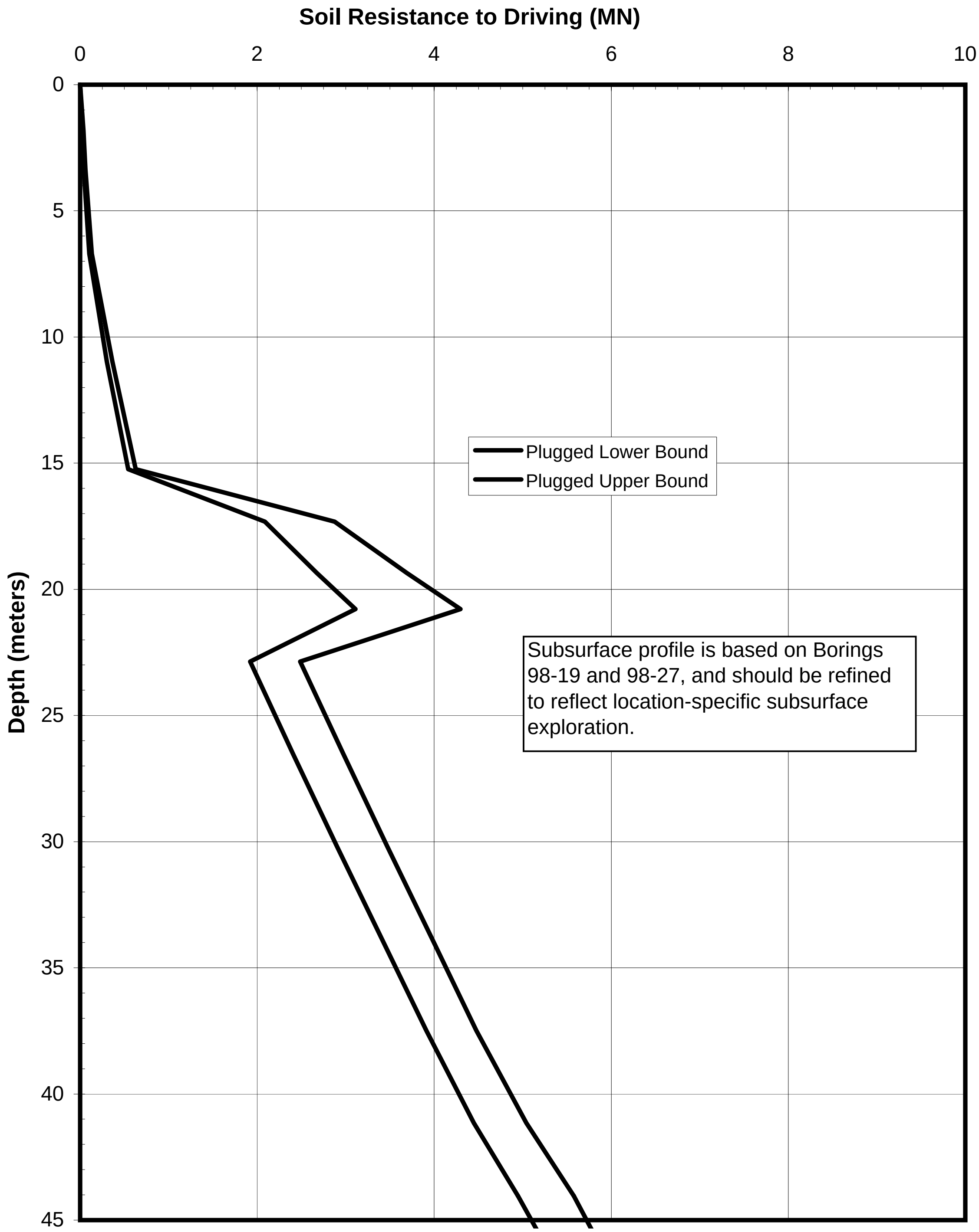
EXAMPLE SOIL RESISTANCE TO DRIVING
Skyway Temporary Tower: BE and BW, Pier E2 to E3
0.41-Meter-Diameter Steel Pipe Pile
SFOBB East Span Seismic Safety Project



EXAMPLE SOIL RESISTANCE TO DRIVING
Skyway Temporary Tower: BE and BW, Pier E2 to E3
0.61-Meter-Diameter Steel Pipe Pile
SFOBB East Span Seismic Safety Project



EXAMPLE SOIL RESISTANCE TO DRIVING
Skyway Temporary Tower: BE and BW, Pier E2 to E3
0.41-Meter-Square Precast Concrete Pile
SFOBB East Span Seismic Safety Project

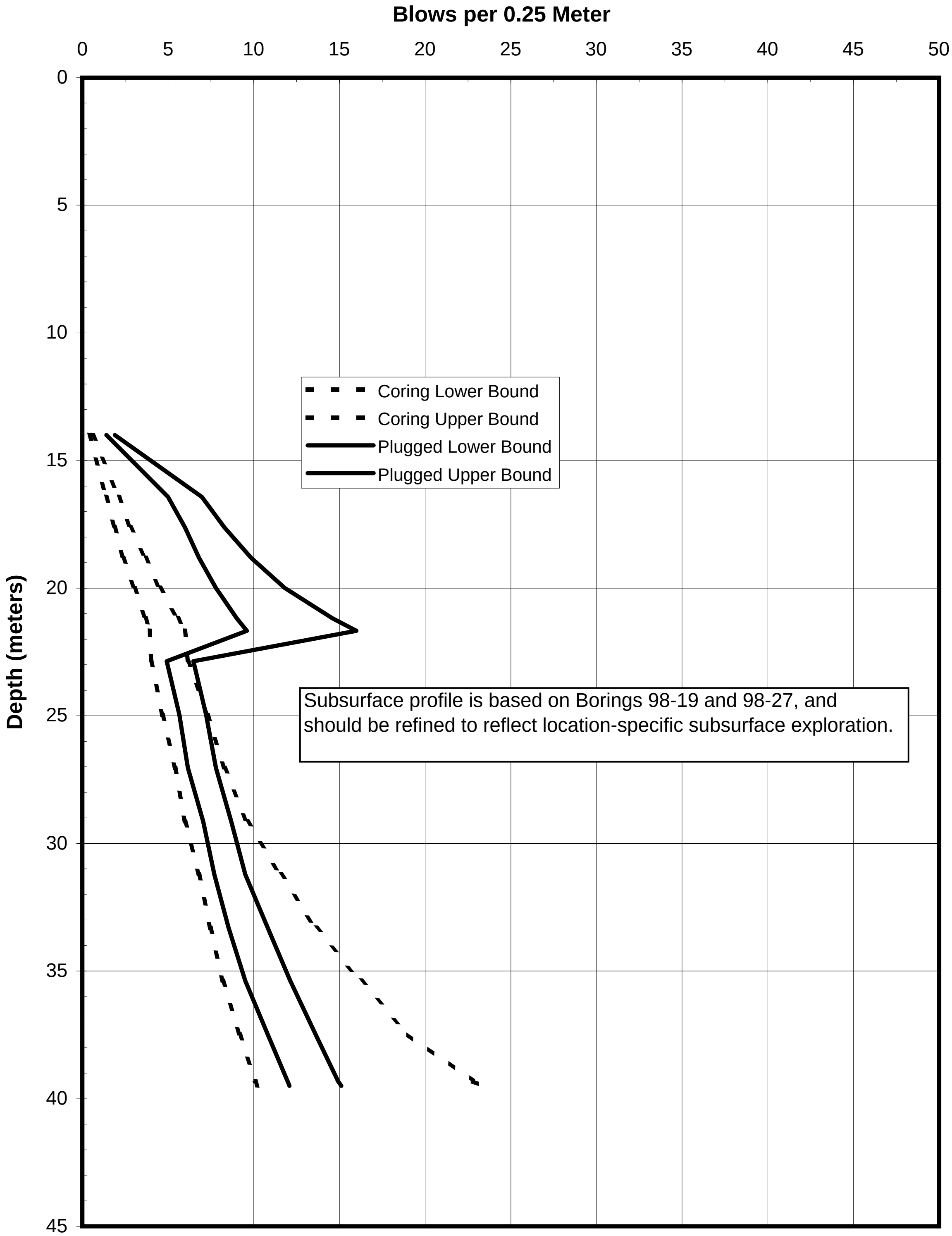


EXAMPLE SOIL RESISTANCE TO DRIVING

Skyway Temporary Tower: BE and BW, Pier E2 to E3

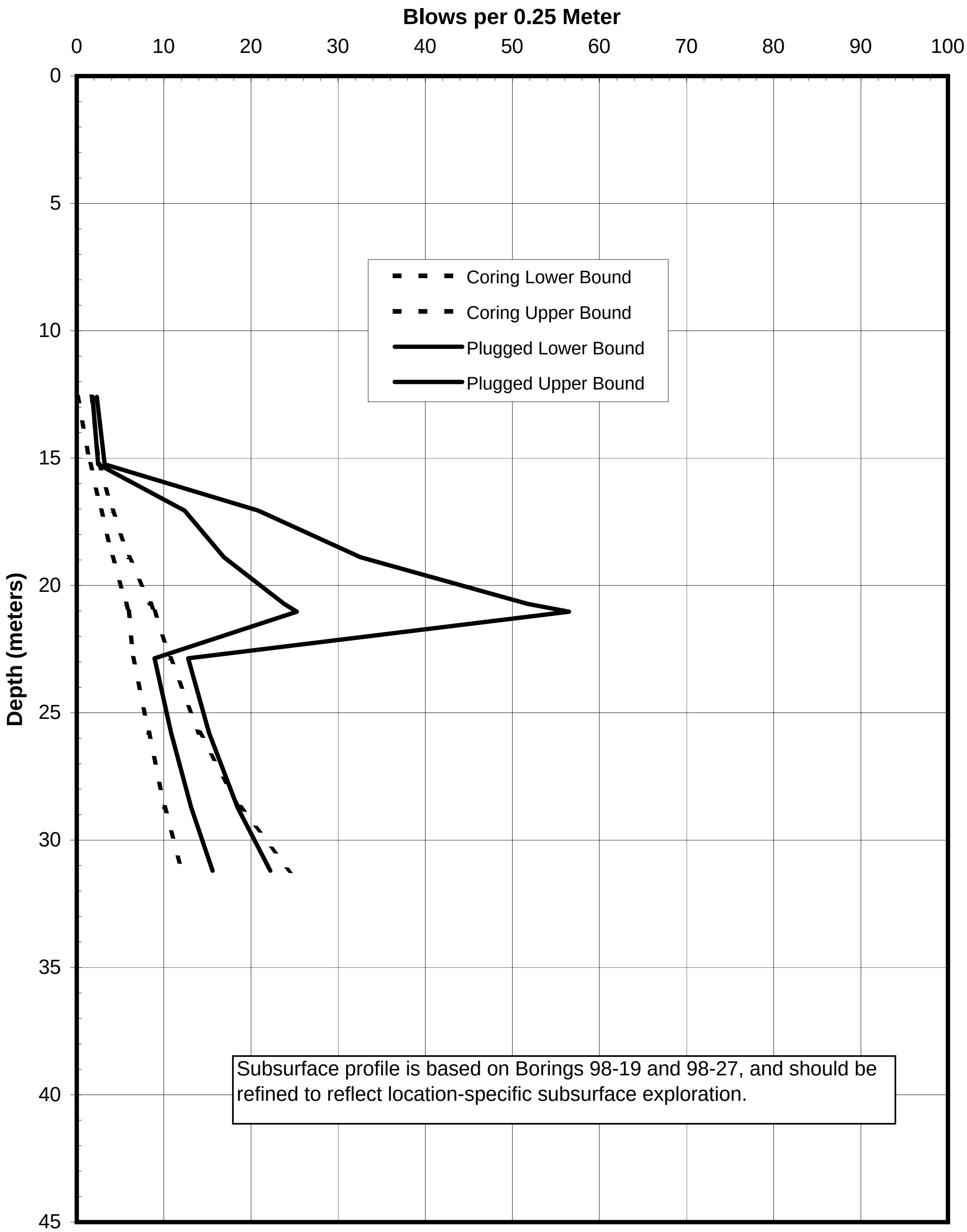
0.61-Meter-Square Precast Concrete Pile

SFOBB East Span Seismic Safety Project

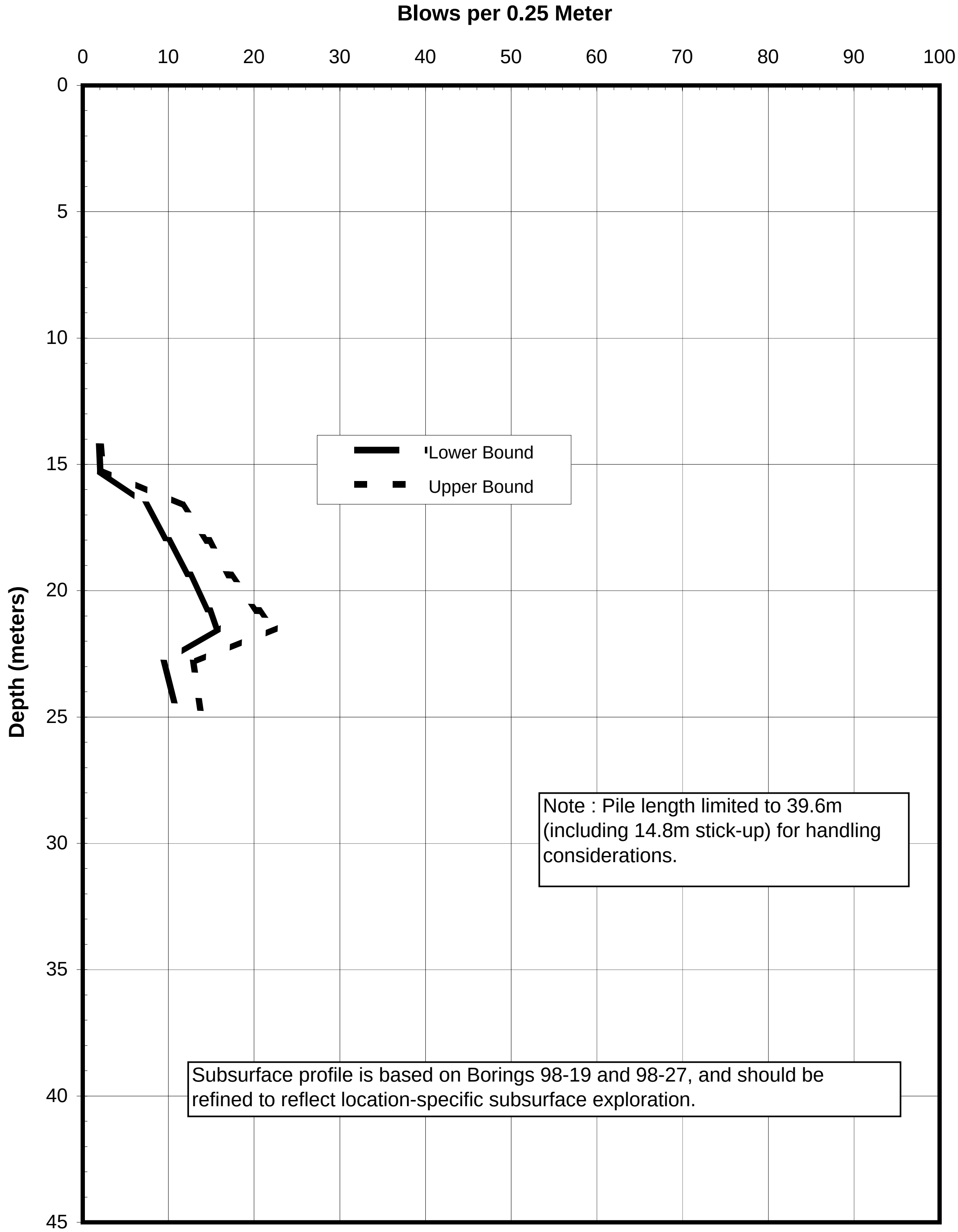


EXAMPLE PREDICTED BLOW COUNTS
Delmag D62 Hammer
Skyway Temporary Tower: BE and BW, Pier E2 to E3
0.41-Meter-Diameter Steel Pipe Pile
SFOBB East Span Seismic Safety Project

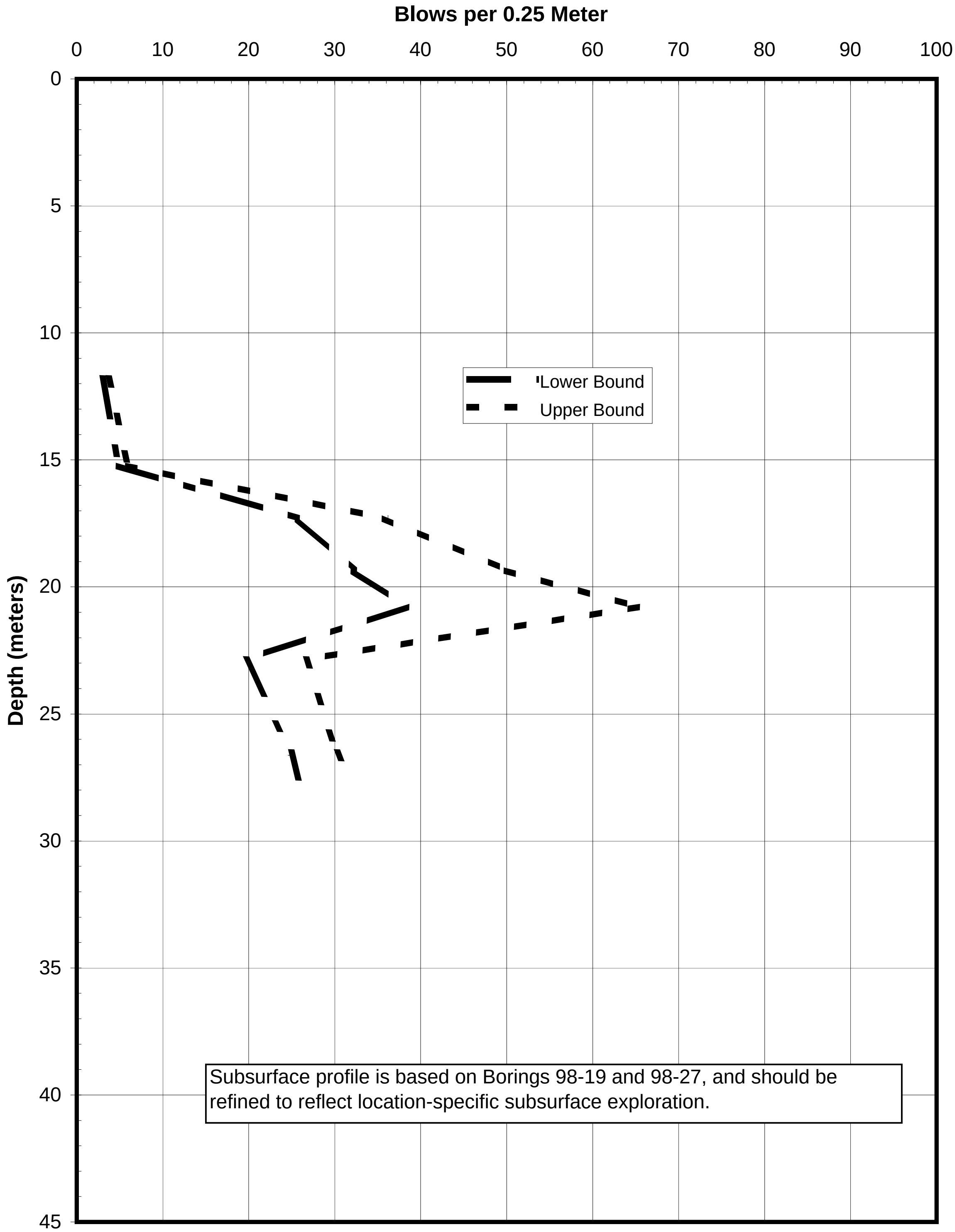
PLATE D2-B.1



EXAMPLE PREDICTED BLOW COUNTS
Delmag D62 Hammer
Skyway Temporary Tower: BE and BW, Pier E2 to E3
0.61-Meter-Diameter Steel Pipe Pile
SFOBB East Span Seismic Safety Project

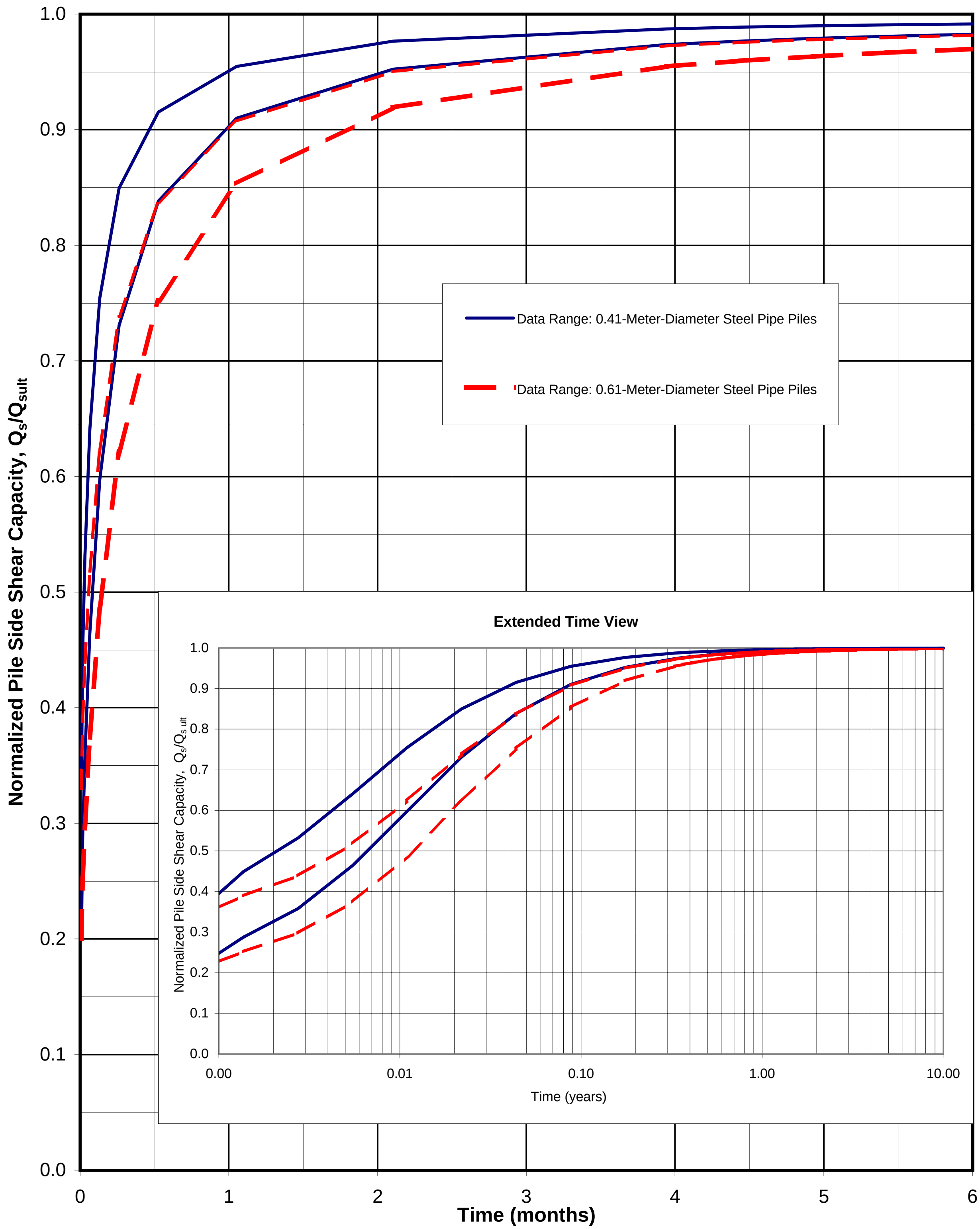


EXAMPLE PREDICTED BLOW COUNTS
Delmag D62 Hammer
Skyway Temporary Tower: BE and BW, Pier E2 to E3
0.41-Meter-Square Precast Concrete Pile
SFOBB East Span Seismic Safety Project



EXAMPLE PREDICTED BLOW COUNTS
Delmag D62 Hammer
Skyway Temporary Tower: BE and BW, Pier E2 to E3
0.61-Meter-Square Precast Concrete Pile
SFOBB East Span Seismic Safety Project

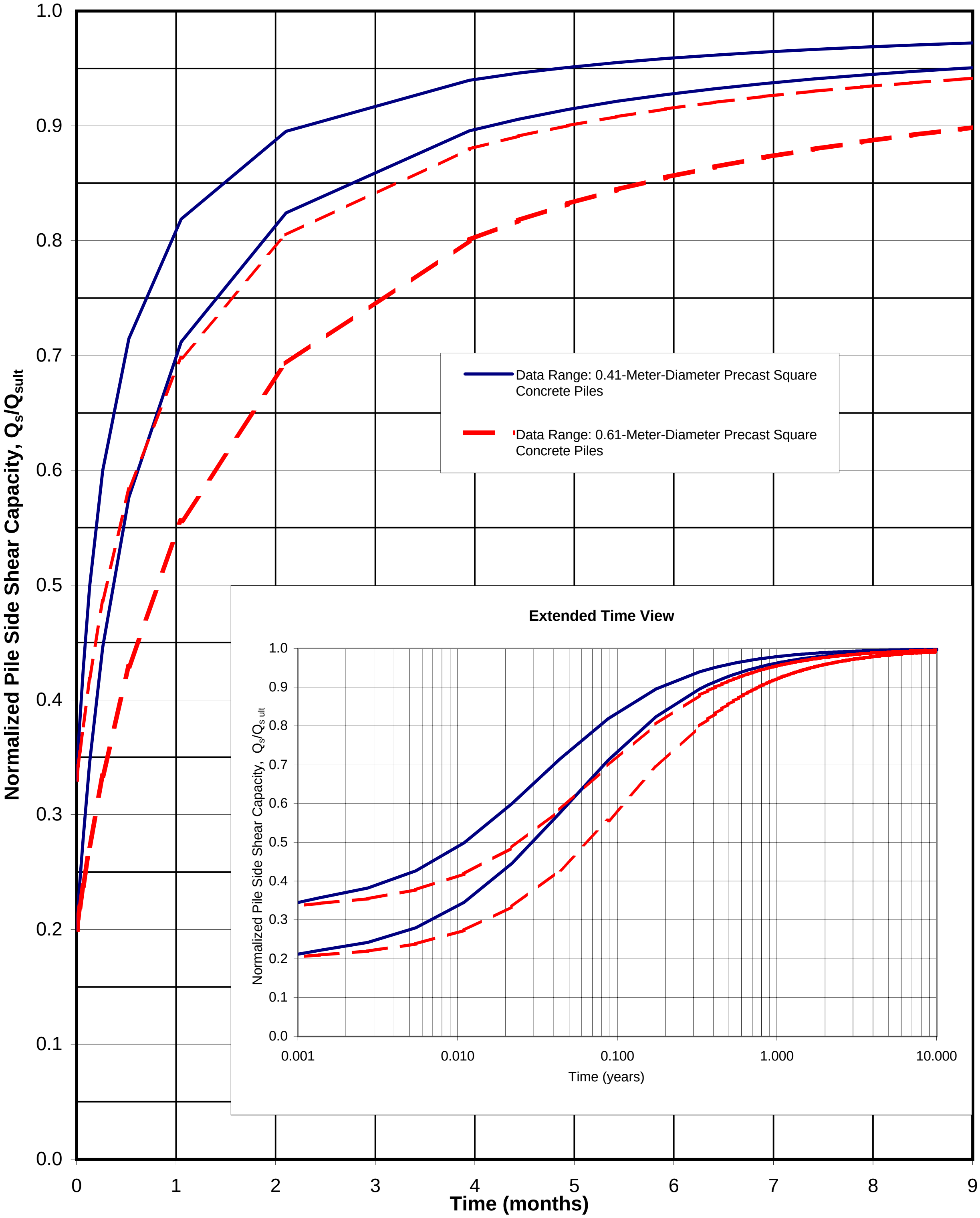
APPENDIX E
SETUP ANALYSIS



PREDICTED SETUP OF SKIN FRICTION IN CLAY

0.41- and 0.61-Meter-Diameter Steel Pipe Piles

SFOBB East Span Seismic Safety Project



PREDICTED SETUP OF SKIN FRICTION IN CLAY
0.41- and 0.61-Meter Precast Square Concrete Piles
SFOBB East Span Seismic Safety Project